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## Civil Engineers

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# AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

## PAPERS

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### DEVELOPMENT OF CBR FLEXIBLE PAVEMENT DESIGN METHOD FOR AIRFIELDS

#### A SYMPOSIUM

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NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by June 1, 1949.

## FOREWORD

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At the beginning of the construction program for World War II, the Corps of Engineers, United States Army, after a survey of available flexible pavement design methods, adopted the empirical method in use by the California Division of State Highways, known as the California Bearing Ratio (CBR) method. This method uses the CBR of the soil, which is an index of strength obtained by a simple shear test, and a set of design curves for determining the total required thickness of base and pavement for a given wheel load.

When the method was adopted, design curves were available for wheel loads as great as approximately 12,000 lb. Since the situation was urgent, these curves were immediately extrapolated to wheel loads of 60,000 lb, and a research program was initiated to provide information for verifying or revising the curves. Furthermore, since the CBR test had not been used widely outside California, a project to study it was included in the program. Many difficulties were encountered in the adaptation of the CBR method to the design of airfield pavements for heavy wheel loads and the research program was extended and revised accordingly.

Various phases of the program were completed during World War II and reports on these phases, which are mentioned in the several papers, were published. Other phases of the program were completed on which formal reports were not prepared. The results of both the published and unpublished investigations have provided design criteria which appear in the *Engineering Manual for War Department Construction*.<sup>1</sup>

The purpose of this Symposium is to present the work that has been accomplished by the Corps of Engineers in connection with the adaptation of the CBR method of design from highway construction to airport construction. In particular, it is desired to paint a true picture of the CBR test and its value as a design tool for flexible pavements. Paper No. 1 gives the background of the adoption of the method, a summary of the study that has gone into the improvement of the method, and the future of the method. Paper No. 2 outlines the development of the original method for highway design. In paper No. 3, the procedures by which the design curves were extrapolated from 12,000 lb to 60,000 lb are presented. Paper No. 4 gives the results of the study conducted on the CBR tests. Papers Nos. 5, 6, and 7 describe, in considerable detail, the three largest accelerated traffic tests that were conducted to obtain data for the design curves and paper No. 8 describes a number of smaller accelerated traffic tests. Paper No. 9 shows the correlation of the CBR curves with the data that have been obtained from the accelerated traffic tests and from studies of actual airfield pavements. Paper No. 10 presents a procedure whereby the design curves for single wheels can be translated into design curves for multiple wheel assemblies of practically any combination. Paper No. 11 is a discussion of the utility of the method. The good features and the shortcomings of the method are indicated and the general applicability is given.

<sup>1</sup>"Airfield Pavement Design," *Engineering Manual for War Department Construction*, Office of Chf. of Engrs., U. S. Army, Washington, D. C., May, 1947, Pt. XII, Chapter 2.

## NOTATION

The letter symbols in this Symposium conform essentially with "Soil Mechanics Nomenclature," ASCE *Manual of Engineering Practice No. 22*. Discussers are requested to use the same notation and to select new symbols accordingly.

$d$  = distance between the inside edges of the tracks made by dual wheels of a test plane;

$G$  = specific gravity;

$I_P$  = plasticity index;

$k$  = modulus of soil reaction;

$N$  = a load concentration factor;

$s$  = spacing center to center of dual wheels; and

$w$  = water content:

$w_L$  = liquid limit;

$w_P$  = plastic limit.

## ACKNOWLEDGMENTS

The CBR test and the CBR method of design for highways were originally developed by the California Division of Highways under the general supervision of T. E. Stanton, M. ASCE, material and research engineer, and under the immediate direction of O. J. Porter, M. ASCE, senior physical testing engineer.

At the beginning of World War II this method was selected by the Corps of Engineers for use in the design of flexible pavements. The responsibility for the development of design criteria for airfield construction was delegated by the Corps of Engineers to the Military Construction Branch, Engineering and Development Division (subsequently Airfield Branch, Engineering Division), Office, Chief of Engineers, under the direction of James H. Stratton, followed by L. C. Urquhart, both Members, ASCE, Henry C. Wolfe, and D. S. Burns. In addition to Gayle McFadden and Thomas B. Pringle, Members, ASCE, chief of the Airfield Branch and chief of the Runway Section, respectively, the early development work was under the responsible direction of Reuben M. Haines, Assoc. M. ASCE, until ill health forced him to resign his position. Consultants to the Corps of Engineers assisted materially in the adaptation of the California method to airfield pavement design for heavy wheel loads. At the time the CBR method was adopted these consultants were A. Casagrande, M. ASCE, T. A. Middlebrooks, Assoc. M. ASCE, and Mr. Porter. Later a formal Board of Consultants was established. Included on this board were the three consultants previously named and H. M. Westergaard and J. L. Land, Members, ASCE, P. C. Rutledge, Assoc. M. ASCE, and F. C. Lang (deceased).

Laboratory, field, and office studies required to verify and modify the California method were primarily the responsibility of the Soils Division, Waterways Experiment Station, Mississippi River Commission. This work was directed by W. J. Turnbull, W. H. Jervis, and W. K. Boyd, Members, ASCE. The studies included a comprehensive investigation of the method of test and application of test results, consolidation of accelerated traffic test section studies, tests on existing airfields, special studies, and the development of CBR design criteria for multiple wheel loads.

## DEVELOPMENT AND SCOPE OF INVESTIGATION

BY GAYLE MCFADDEN<sup>2</sup> AND THOMAS B. PRINGLE,<sup>3</sup>  
MEMBERS, ASCE

During the latter part of November, 1940, the responsibility for the design and the construction of military airfields was assigned to the Corps of Engineers, United States Army. Later the same function pertaining to civil aeronautics airfields, of military occupancy, was delegated to this same organization.

It soon became apparent, for a variety of reasons, that a simple, practical, as well as a uniform method of design would be necessary. For example, the prior experience of the majority of the Corps' engineers had been generally confined to river and harbor, flood control, and seacoast fortification. This explains the necessity for the elimination of a diversity of designs based on judgment of personnel not fully conversant with the requirements of airplane traffic. Other reasons for seeking a simple, practical, and uniform method of design were: (a) To obviate the use of untried methods; (b) to insure adequately designed pavements; (c) to provide a method not subject to variation occasioned by arbitrary cost differentials of local and competitive materials; (d) to avoid reductions of pavement thickness in order to balance cost; and (e) to establish a procedure for further development of design methods through tests, investigations, and study of the actual behavior of pavements.

The Westergaard method of design of rigid type (concrete) pavements seemed fairly universal in its application and was quickly adopted by the Engineer Department as a standard of design for that type of pavement.

The selection of a design method for flexible pavements presented a more difficult problem. There were several methods in vogue that had been developed primarily for highway design. All these were studied and all had considerable merit; but most had some apparent fallacy that seemed to defeat the aforementioned objectives.

In most of the methods in current use at that time, the basic design assumption was the bearing capacity of the subgrade. The question was raised as to whether the procedures for determining this value, as related to flexible pavements, had been adequately determined.

Since it was desirable to select a method that could be adapted with the least delay, and since the selection of one in common usage seemed the most expeditious manner, personnel of the Corps of Engineers performed several special field investigations (Langley Field (Virginia), Bradley Army Airfield (Connecticut), and a Virginia Highway Department test section) to try and develop the procedures for determining the subgrade bearing capacity values in order that they might be used in the theoretical formulas of the several methods.

<sup>2</sup> Head Engr., Chf., Airfields Branch, Eng. Div. (Military Constr.), Office, Chf. of Engrs., Dept. of the Army, Washington, D. C.

<sup>3</sup> Prin. Engr., Chf., Runway Section, Airfields Branch, Eng. Div. (Military Constr.), Office, Chf. of Engrs., Dept. of the Army, Washington, D. C.

The results of these special field investigations indicated that:

1. The length of time required to develop a satisfactory test procedure would preclude its use in the war emergency program then being faced;
2. In the field plate-bearing test, the proper deflection to determine the "bearing capacity" depends on the basic assumptions in the formula and varies according to combinations of many factors; and
3. In most cases the results of the plate-bearing test would not be applicable to the soil-moisture conditions expected ultimately to develop below a pavement, and it would be difficult to develop a satisfactory method for adjusting the test results to the various moisture conditions.

Some methods contemplated using plate-bearing tests on pavement surfaces for the design of flexible pavements. Field tests made on pavements and base courses led to the conclusion that the same factors which must be considered in using this test for subgrades must also be considered for pavements. In addition, the compressibility of the pavement and that of the base material entered into the problem. These conclusions were considered applicable primarily to flexible pavements because, in this case, the stress distribution from load to pavement, base and subgrade, results in shear failure; and it is practically impossible to differentiate between the deflection under plate-bearing tests due to consolidation and plastic deformation. These conclusions naturally do not hold true in designing rigid type pavements since the modulus of soil reaction  $k$  of the subgrade is essentially a measure of consolidation, and service behavior has demonstrated that a plate-bearing test is satisfactory for determining its value. The logical inference follows that a shear test is proper in the design of flexible type pavements.

Further studies by the Corps of Engineers indicated that the stress conditions and the behavior of subgrades and the base courses under repetitive wheel loads were extremely complex and could not be determined by a single test.

From these studies a definite conclusion was reached that some method other than the application of a theoretical formula would have to be used, at least during this period of pressing need. Therefore, adaptation of an empirical method that had been proved for highway loading appeared to be the only solution. This decision narrowed the field; and, after some months of investigation of suggested methods, the principles used by the California Highway Department in designing flexible pavements for highway loading, known as the CBR method, were adopted tentatively.

The controlling reasons for the adoption were many. Among these reasons were: (1) The CBR method had been correlated to the service behavior of flexible pavements and construction methods and successfully used by the State of California for a number of years; (2) it could be more quickly adapted to airfield pavement design for immediate use than any other method; (3) it was thought to be as reasonable and as sound as any of the other methods investigated; (4) two other states were known to have methods of a similar nature that had been successful; (5) the subgrade modulus could be tested with simple portable equipment either in the laboratory or in the field; and (6) testing could

be done on samples of soil in the condition representative of the foundation-moisture state under most pavements.

The selection of the method was only the first step in its assimilation by the Corps. A lecture course was organized and conducted in the United States Engineer Office, Sacramento, Calif., on the "California Method of Determining the Relative Bearing Value of Soils and Its Application to the Design of Highways and Runways." The scope of the course was limited to the design of flexible type pavements. Its purpose was to provide an intensive short course to acquaint personnel of the Corps of Engineers with the CBR method. The course was attended by representatives of the then eleven regional offices of the Corps, known as Division offices; representatives of the Office of the Chief of Engineers; and representatives of what was later to become the Flexible Pavement Laboratory. The representatives were in turn to instruct the personnel in the subordinate offices of the Divisions (known as District offices) in order that application of the procedure could be made rapidly available.

The adoption of the method did not stop with the familiarization course given the personnel of the Corps of Engineers. An exhaustive study was initiated and has become a continuing program in an attempt to obtain and to maintain the maximum degree of beneficial use from the method.

Since the CBR method is empirical and its design curves are used in connection with a modulus of shearing resistance of soils determined by a special CBR test, it was considered necessary to investigate this test to develop a correlation with field performance for airfield construction similar to that already accomplished by the California Division of Highways for road work. This study was authorized by the Office of the Chief of Engineers, in September, 1942, for accomplishment by the Waterways Experiment Station, Vicksburg, Miss.

The scope of the study was expanded as it became obvious that there were further fields to explore. Interim reports were published from time to time for dissemination to the field offices of the Corps and to provide information in the preparation of the *Engineering Manual for War Department Construction*. After approximately 3 years of study, the investigation culminated in a comprehensive final report.<sup>4</sup>

When the CBR method was first adopted, the design curves for total pavement thickness requirements had only been correlated with field performance sufficiently to accommodate wheel loads normally found in highway usage. Airplane wheel loads generally began where highway loads ended. Therefore, it was necessary to provide an extension of these curves. The best talent available, including men familiar with the development of the original curves, was set to work to evolve a tentative extrapolation to fulfil an immediate requirement. These extrapolations were published as "tentative" design curves pending their validation.

Work was begun immediately to provide the necessary correlation with field performance under military airplane loads to validate the extrapolations.

<sup>4</sup>"The California Bearing Ratio Test As Applied to the Design of Flexible Pavements for Airports," Technical Memorandum No. 213-1, U. S. Waterways Experiment Station, Vicksburg, Miss., 1 July 1945.

This operation was accomplished by determining the CBR of the subgrade and base and performing accelerated traffic tests (a) on existing pavements whose construction history was known and (b) on specially constructed full-scale test sections where all construction items were carefully controlled. In all tests, airplane traffic was simulated as closely as possible with regard to load and tire characteristics. In the specially built sections, the various items were intentionally overdesigned and underdesigned to bracket the results and to give the widest range of information. Post-traffic examination of surface and sub-surface conditions was made and recorded.

From the plot of the test results, the design curves were modified where necessary and this modification and other pertinent design information developed from the tests were immediately made available to the designers of the Corps of Engineers and incorporated into the *Engineering Manual for War Department Construction*.

One or two traffic tests were not considered satisfactory to develop the required information. Not less than four were run on existing pavements at Corpus Christi, Tex.; Dothan, Ala.; Fargo, N. Dak.; and Lewistown, Mont. Not less than six specially built test sections were subject to traffic—among which were those at Langley Field, Virginia; Stockton Army Airfield, California (Sections Nos. 1 and 2); Eglin Field, Florida; Barksdale Field, Louisiana; and Marietta Aircraft Assembly Plant, Georgia. The wheel loads ranged from 15,000 lb to 200,000 lb. The traffic was generally continued until the behavior of the pavement indicated failure, incipient failure, or satisfactoriness.

A detailed account of the studies and tests that have gone into the improvement of the CBR method and the CBR test, to determine the modulus of shearing resistance of soils, is contained elsewhere in this Symposium.

It is recognized that the CBR method may not supply the ultimate answer in flexible pavement design. Pavement failures have occurred in cases where the design was apparently adequate according to the CBR method; however, these instances are in minority. The enthusiastic reports from engineers in the theaters of operation, where speed and simplification of design were mandatory, the ease with which the method was assimilated in construction in the continental limits of the United States, and the growing demand from the engineering profession in the United States and other countries for further information on the developments made by the Corps of Engineers indicate that the CBR method has a definite place in flexible pavement design for some time to come. It was considered the most desirable available method at the time of adoption, and the studies and the testing that have gone into its improvement, together with growing uniformity in laboratory test procedures, have maintained it as the most desirable method to date.

However, the Corps of Engineers is not inseparably "wedded" to the CBR method. Continuing studies have as their purpose the development of basic understanding of the stresses and the strains that occur in subgrades, bases, and wearing courses under wheel loads, and the development of test procedures to measure behavior characteristics. The results of these studies, together with data being obtained on the effect of moisture changes and other weather

features, may permit the development of a more rational method of design. However, results to date (1948) indicate that such a method may not be available for quite some time. When a more rational method is developed, by the Corps of Engineers or others, the superiority of which can be proved and which contains required features of simplicity of testing and application, there is no doubt that its adoption by the Corps is a foregone conclusion. Until this advance occurs, the Corps of Engineers is continuing to use and to develop the CBR method.

## DEVELOPMENT OF THE ORIGINAL METHOD FOR HIGHWAY DESIGN

BY O. J. PORTER,<sup>5</sup> M. ASCE

During 1928 and 1929, the California Division of Highways made an intensive investigation of pavement failures throughout the state. Areas that had failed were investigated to determine the local drainage conditions and other factors affecting the stability of the pavement. Holes were drilled through pavements and undisturbed samples were obtained which would furnish information as to density and moisture content of the materials underlying the pavement surface. These investigations pointed to three principal types of pavement failure: (1) By lateral displacement of the subgrade material as the result of the pavement having absorbed water and softened after construction; (2) by differential settlement of materials underlying the pavement; and (3) by excessive deflection of the materials underlying the pavement, resulting from repetitions of load.

Types (1) and (2) of failure could often be traced directly to poor compaction during construction. Although poor drainage conditions contributed to the rate of increase of moisture beneath the pavement, the ultimate moisture conditions were similar regardless of drainage conditions. Examination of pavements which did not fail indicated an increase of moisture after construction, but the extent of the moisture increase was limited by the degree of compaction of the material. Pavement conditions were found to be more dependent on compaction than on other factors unless, of course, the material showed excessive swelling in the presence of water. Usually type (3) of failure was brought about by insufficient thickness of pavement and by the existence of a base course over soils inherently weak in shear strength—because of either the nature of the soil or poor compaction. Investigation also showed that identification of the soil type did not provide sufficient assurance that a soil was, or was not, a satisfactory subgrade material, since the same soil would behave differently under different conditions of moisture and density. \*

A method of test was sought, therefore, which would be satisfactory for establishing the density that should be used in the construction of subgrades and the shear strength that was required. The experience gained from the study of failures also was helpful in determining the thickness of pavement and base course that would be required over the various soils, taking into account the condition in which it was placed.

To predict the probable behavior of materials underlying the pavement, it was desired to have a simple and quick test that would evaluate the quality of subgrade, subbase, and base course materials. For this purpose, static field load tests were first tried. These tests were influenced to a large extent by the elastic and the plastic deformation of the material beneath the test load. The

<sup>5</sup> Cons. Engr., Sacramento, Calif.; Consultant to the Office, Chf. of Engrs., Dept. of the Army, Washington, D. C.

extent of each of these types of deformation varies greatly with the moisture and the density conditions in the soil beneath the test area. It was practically impossible to moisten the soil properly in the field to the depth affected by the load test; and, as a consequence, the results of such tests might represent a test condition not related to the ultimate condition and the action of the subgrade during service.

A bearing ratio test was devised in 1929 in an attempt to eliminate some of the objections to field loading tests and to provide a quick method for comparing local base and subbase materials available for reinforcing the subgrade. This test has been called the CBR test. An empirical relationship was subsequently established between the test value and the suitability of base and subgrade materials. Samples of the material to be investigated were first thoroughly consolidated to the density obtainable under good construction methods or to approximately the density ultimately produced by traffic in good subgrade materials. This procedure eliminated, to a very large degree, most of the consolidation deformation that often influences static load tests made in the field. The compacted specimen was next soaked for 4 days under a surcharge representing the weight of the pavement, to permit the specimen to swell and absorb moisture, with consequent loss of strength. The CBR test was then made on the specimen to determine the resistance to lateral displacement, thus measuring the combined influence of cohesion and internal friction. Fig. 1 illus-

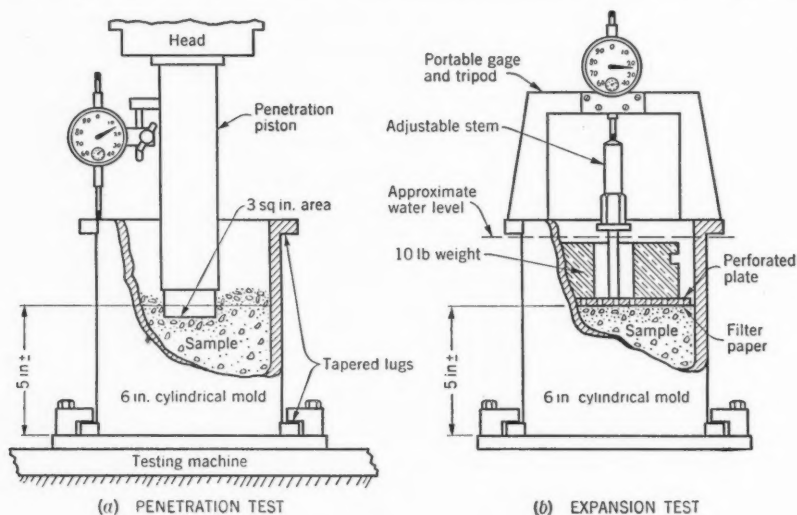


FIG. 1.—PENETRATION AND EXPANSION TEST, CBR METHOD

trates the penetration and the expansion (soaking) parts of the CBR tests. The resistance to penetration in the test is expressed as a percentage of the resistance for a standard crushed stone. Tests were performed on a large number of typical crusher-run materials which were deemed representative of high-quality base course materials. The average of these results was taken as a CBR of 100% and results of tests on other materials were expressed as a per-

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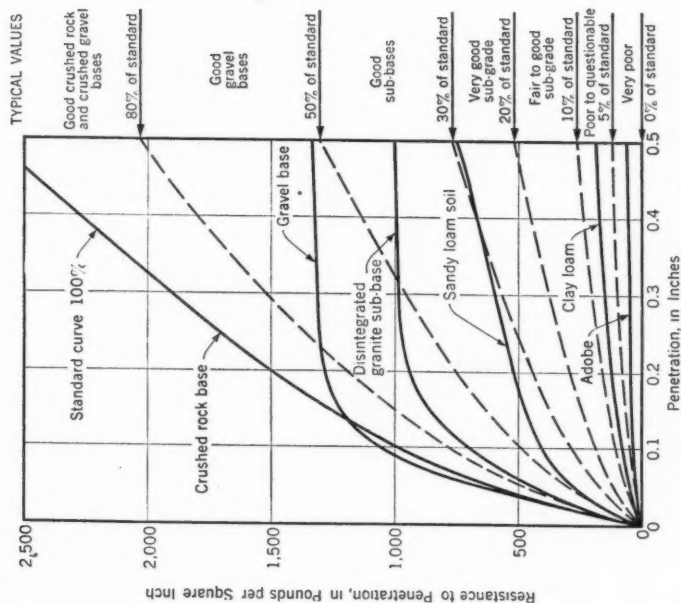


FIG. 2.—RESISTANCE TO PENETRATION, COMPACTED AND SOAKED SPECIMENS

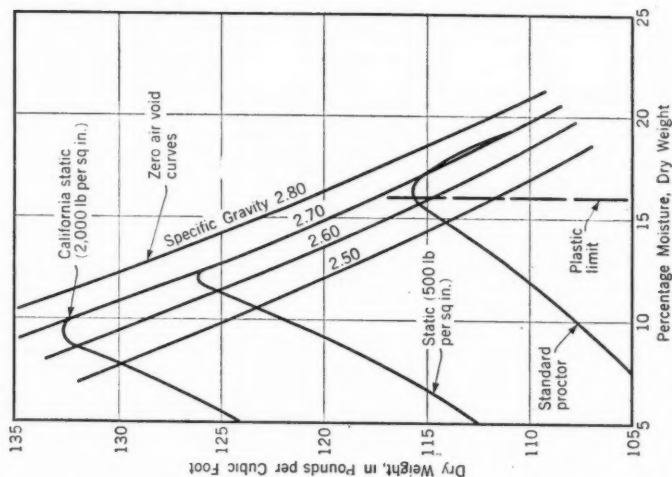


FIG. 3.—MOISTURE-DENSITY RELATIONSHIP, CLAY-LOAM SOIL

centage of the 100% value. Typical values for a number of soil and base course materials are shown in Fig. 2.

As the increments of penetration are large with respect to the loaded area, the results are influenced only to a minor degree by elastic deformation. In view of the fact that it is not practical to test large specimens in the laboratory and to penetrate the test specimens over an area comparable in size to field loading conditions, the results are influenced to a considerable degree by cohesion and interlocking of particles. Therefore, a surcharge on the area adjacent to the penetration piston is necessary to develop friction in clean, round-grained sand in which particle interlocking and cohesion are low. The results of the penetration tests on such materials without a surcharge should be interpreted as representing conditions in which the material is not confined by the weight of a pavement. The tests with a surcharge indicate the increased stability of the material when used in the lower portion of foundation courses where the confining weight of the overlying pavement and the base course increase the frictional resistance.

It has been noted in a preceding paragraph that the CBR test is performed on a soil specimen that has been compacted to a density equivalent to that obtained in good construction or to that obtained by satisfactory subgrade soils after consolidation by traffic. During studies made by the California Division of Highways in 1928 and 1929, subgrade materials which had proved satisfactory in service were compacted in the laboratory under static loads ranging from a few hundred pounds per square inch to 10,000 lb per sq in. and, in some cases, to 20,000 lb per sq in. On the basis of these preliminary tests, a static load of 2,000 lb per sq in. was adopted after it was found that this pressure was necessary to produce a density equivalent to that present under old highways which had been subjected to traffic for a period of years. It was evident not only that the high density resulting from this load was desirable, but also that it could be obtained in the field with proper moisture control and thorough rolling whenever the subgrade and subbase materials were not of an extremely adverse type. Although this static load results in a density that cannot be readily duplicated in the field with heavy clays, the method proved to be both desirable and practical for control of embankment construction.

Briefly, the compaction test adopted consisted of placing a sufficient amount of the material under test in a 6-in.-diameter mold and compacting it under a static load of 2,000 lb per sq in. using a standard laboratory compression machine. The load was applied slowly at a rate of 0.05 in. per min and was maintained for a period of 1 min, and then gradually released. The density was determined by the usual laboratory methods. Compaction of several samples at varying water contents would yield different densities, and the optimum moisture content was taken as the moisture content at which the greatest density was obtained. For field control, where a compression machine was not available, an impact method of compaction using twenty blows of a 10-lb tamper with an 18-in. drop on each 1-in. layer was adopted. Thus, a sufficiently accurate duplication was obtained for field use of optimum moisture and density as determined by the static test.

Fig. 3 shows the relationship between the static and the standard Proctor compaction tests obtained on a clay-loam soil which is classified as A4 by the Public Roads Administration (PRA) classification or as CL by the Casagrande classification.<sup>6,7</sup> (Other characteristics of the soil were: CBR, 8.0%; expansion, 5.3%; liquid limit ( $w_L$ ), 23%; plastic limit ( $w_P$ ), 16%; and plasticity index ( $I_P$ ), 70%. Its component parts were: Sand, 48%; silt, 26%; and clay, 26%.)

Expansion during the soaking period indicates the tendency of the materials to swell when they are allowed access to water. The uplift, unless absolutely uniform, may roughen and crack nonrigid pavements. Because this volume change is seldom uniform throughout the length of a roadway or runway, the surface drainage may also be disrupted. Subgrades which have proved satisfactory usually show an expansion of less than 3% of the initial height of the specimen during the 4-day soaking period whereas good base and subgrade materials show less than 1%. Many of the poorer clay and adobe soils often show an expansion ranging from 7% to 20%. Studies of soil expansion have indicated that each material has a state of equilibrium regarding density and moisture content for each condition of confinement and that the moisture capacity of a soil varies inversely with the superimposed load. As an example, tests on an adobe that was soaked until all expansion ceased are demonstrated by Fig. 4.

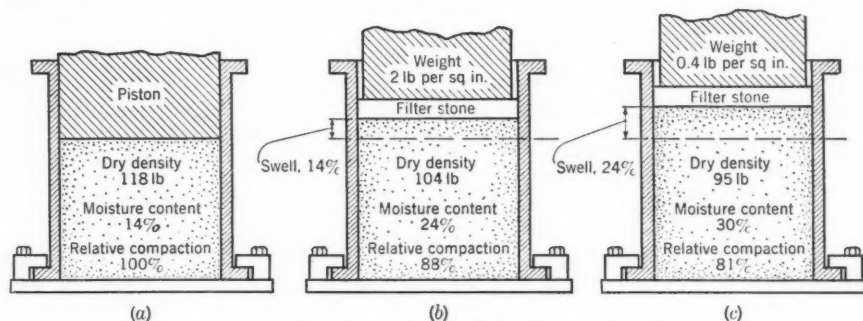


FIG. 4.—EXPANSION OF ADOBE SOIL WHEN SURCHARGED, WITH TWO PAVEMENT AND SUBBASE LOADS

Fig. 4(a) represents the sample before soaking, when the soil has been compacted to its maximum subgrade density. Figs. 4(b) and 4(c) represent the sample soaked until expansion ceased. In Fig. 4(b), the surcharge (2 lb per sq in.) is assumed equal to the weight of a 6-in. concrete pavement and a 1-ft gravel base, whereas the surcharge in Fig. 4(c) (0.4 lb per sq in.) is equal to the weight of a 5-in. concrete pavement only. Fig. 4(b) indicates that the material will swell 14% with an increase in moisture content from 14% to 24% under 2 lb per sq in. whereas under 0.4 lb per sq in. the swell will be 24% with an increase in moisture content from 14% to 30%.

<sup>6</sup> "Classification and Identification of Soils," by Arthur Casagrande, *Proceedings, ASCE*, June, 1947, p. 792, Table 3.

<sup>7</sup> *Ibid.*, pp. 794-797, Table 4.

Studies also indicated that the expansion varied with the quantity of air contained in the material. At a given density, minimum expansion occurs when the voids are approximately full of water. It is desirable, therefore, that expansive types of soils be compacted at a density near the no-swell state and at a moisture content that will approach minimum air voids for this density. The maximum density at which the material will not expand an excessive amount is also desirable to secure the highest possible stability.

Investigations made from 1928 to 1942, on both adequate pavements and flexible pavements that failed, furnished considerable empirical data for correlation of the CBR requirements with service behavior. From these data, curves were formulated for determining the thickness of pavement and base course required to carry traffic adequately over compacted subbase and subgrade materials.<sup>8</sup> Curves A and B, Fig. 5, show the minimum thickness of reinforce-

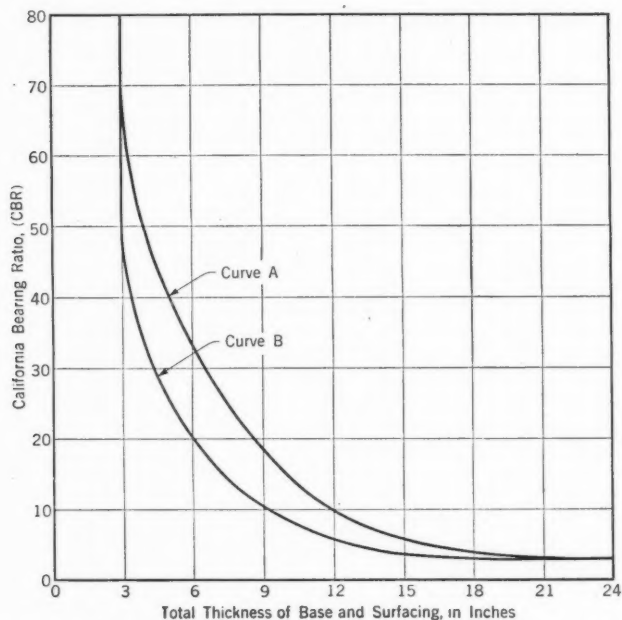


FIG. 5.—TOTAL THICKNESS OF BASE AND SURFACING IN RELATION TO CBR VALUES

ment used in 1942 for light and medium heavy traffic on the California highway system. Curve A was developed on the basis of current experience and is believed to be adequate generally for average traffic conditions. Curve B represents values developed from the original survey of the Materials and Research Department of the California Division of Highways. It demonstrates that foundation conditions under the pavement and the surfacing may be considered satisfactory at the time of the survey. These latter values are con-

<sup>8</sup> "Designing Foundation Courses for Highway Pavements and Surfaces," by Fred J. Grumm, *California Highways and Public Works*, March, 1942, p. 6.

sidered a minimum requirement on roads with light traffic. With the help of these curves pavements are designed by securing samples of the proposed materials, and by preparing and testing them in accordance with the procedures described in the preceding paragraphs. The bearing ratio obtained from the test is then projected horizontally from the left-hand side of Fig. 5 until it intersects the design curve applicable to the project. The thickness of pavement and base required above the soil tested is then read from the bottom scale directly below the point of intersection. The same procedure may be repeated for one or more successively stronger base materials, thus developing a pavement design having a layered base construction. The minimum thickness of surfacing required is 3 in., and no surfacing is laid directly on a base with a bearing ratio of less than 50% (see Fig. 5).

## ADAPTATION TO THE DESIGN OF AIRFIELD PAVEMENTS

BY T. A. MIDDLEBROOKS,<sup>9</sup> ASSOC. M. ASCE, AND G. E. BERTRAM<sup>10</sup>

In the development of the design method based on the use of the CBR test, the California Division of Highways tested adequate sections of flexible highway pavements and sections that failed and correlated the service behavior of the pavement with its bearing ratio. These data were consolidated into a design chart as explained in paper No. 2. This design chart shows the total thickness of base and surfacing required for a flexible highway pavement on a subgrade with a given CBR value. On this chart (see Fig. 5), as stated by Mr. Porter, curve A represents a total thickness of pavement and base course above the subgrade adequate for average highway traffic conditions, and curve B indicates the total thickness considered to be the minimum requirement for light highway traffic.<sup>11</sup>

The empirical nature of the tests made it possible to adapt the CBR equipment and procedures with only minor changes; however, to make use of the design chart, it was essential to evaluate curves A and B in terms of airplane traffic. As drawn on the highway chart, the lines represented a large number of repetitions of highway traffic moving in narrow lanes. In converting these data to equivalent airplane wheel loads, the relationships between the factors—

- a. Relative weights of loaded vehicles;
- b. Characteristics of airplane and automobile tires; and
- c. Lateral distribution of traffic

—were considered. As it was believed that curve A, Fig. 5, was the most reliable, it was used as a basis for the conversion. Although the highway curves were originally drawn for lighter wheel loads, it was known from service behavior of the pavements that 9,000-lb truck wheel loads were supported without distress throughout the life of the pavement. Highway loadings were carried on tires with a deformation of less than 10% whereas data for airplane tires indicated a general deformation of about 35% which resulted in a larger area of contact for any given load. Highway traffic was considered as moving essentially in the same narrow lane whereas the limited information available indicated that runway traffic was fairly well spread over the width of the slab with about 50% of the traffic concentrated in the center third of the pavement. On the basis of these factors, recognizing that the limited data would necessarily need to be supplemented by further tests and observations, it was decided that curve A, Fig. 5, could be assumed to represent a 12,000-lb airplane wheel

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<sup>11</sup> "Designing Foundation Courses for Highway Pavements and Surfaces," by Fred J. Grumm, *California Highways and Public Works*, November, 1941, p. 4.

load. Curve B was judged on the same basis to represent a 7,000-lb wheel load since this was the approximate wheel loading of training planes and represented the lightest traffic requirement for airfields.

Following the establishment of values for the basic curves, wheel load graphs for heavier airplane loadings were determined by extrapolation. The program of static load bearing tests conducted by the Corps of Engineers had shown that the deformation, under wheel load, of an adequately designed flexible pavement is made up of three factors—settlement of the subgrade, compaction of the base and the surface, and elastic deformation. Shear deformation was eliminated because, in a satisfactory pavement, the shearing stress does not exceed the shearing strength. Service behavior records of adequate pavement had indicated that it was necessary for elastic deformation to govern over an extensive period of use. Accordingly, the Office of the Chief of Engineers decided to develop empirical curves by extrapolating the original data on the basis of the elastic theory. Since all bearing tests (even those conducted on confined specimens such as the CBR) are essentially shear tests and since shear deformation must be eliminated in a satisfactory pavement, shear stresses were used as a guide in making the extrapolation.

A review of airplane tire data indicated that a uniform tire pressure of 60 lb per sq in. could be selected for the entire group of planes in use. As B-17's and B-24's were the prevalent types, wheel loads of 25,000 lb, 40,000 lb, and 70,000 lb were selected to cover the range of heavy aircraft loads. Contact areas were computed from wheel loads and tire pressures. Circular areas were used

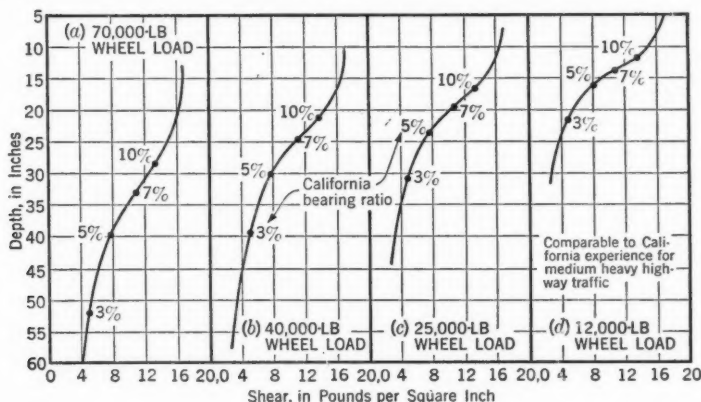


FIG. 6.—EXTRAPOLATION OF HIGHWAY PAVEMENT THICKNESSES BY THE ELASTIC THEORY (TIRE PRESSURE, 60 LB PER SQ IN.)

for ease of computation and also because the difference in shear stresses in the base course and subgrade did not vary materially for elliptical and circular areas. Shear stresses were computed as shown in Fig. 6 by the use of the stress tables.<sup>12</sup> The thicknesses of base course and pavement corresponding to bearing ratios of 3%, 5%, 7%, and 10% were located on the stress curve

<sup>12</sup> "Application of Elastic Theory and Plasticity to Foundation Problems," by Leo Jürgenson, *Journal, Boston Soc. of Civ. Engrs.*, July, 1934, p. 242.

for the 12,000-lb load curve and the stresses corresponding to these thicknesses were noted. On the basis that these stresses should not be exceeded for other wheel loads to retain a uniform standard of design, the stress values were located on the curves for the 25,000-lb, 40,000-lb, and 70,000-lb wheel loads (Figs. 6(c), 6(b), and 6(a)). (The values indicated by the curve in Fig. 6(d), for the 12,000-lb wheel load, were comparable to the California experience for medium heavy highway traffic.) The thickness corresponding to these stresses was transferred to the graph of thickness versus bearing ratio, and curves similar to those shown in Fig. 7 were drawn.

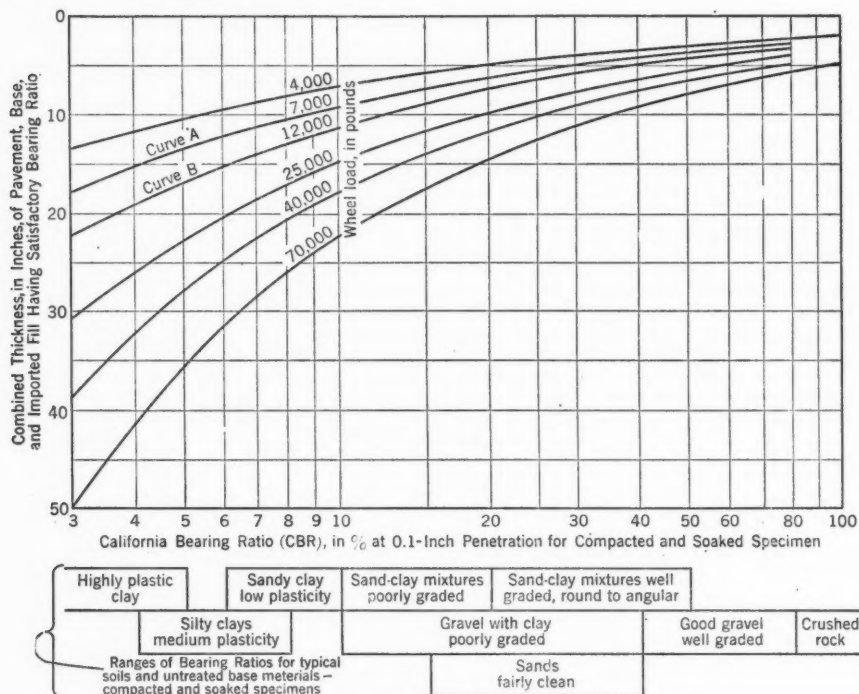


FIG. 7.—TENTATIVE DESIGN OF FOUNDATIONS FOR FLEXIBLE PAVEMENTS

The preliminary design curves obtained by this extrapolation were presented to a meeting of consultants in Washington, D. C., which included engineers from the Office of the Chief of Engineers, Mr. Porter, who developed the California method for highway pavements, and Arthur Casagrande, M. ASCE. The consultants had each made independent calculations to extrapolate the basic curves. Those of Mr. Porter were based on an allowable deformation for the various loads, whereas those of Professor Casagrande were based on the relationships between the relative size of the loaded areas. The three sets of computations were in substantial agreement when the data were reviewed by the group. It was decided that the average thicknesses shown by the three extrapolations were reasonable for the low CBR values; however, the majority

of the members agreed that the less conservative values should be chosen for the higher CBR values. Based on the aforementioned extrapolations and conference decisions, tentative design curves were drawn, as shown in Fig. 7, which expressed the best judgment of the engineers from the Office of the Chief of Engineers, and of the consultants.

At this same meeting an initial program of field and laboratory testing was drawn up to check the tentative design curves immediately, before they were used for design of airfield pavements. The tests are described in subsequent papers; however, it is not amiss to state that, in general, the results of the studies have substantiated the tentative design curves and no appreciable changes in the design curves have been necessitated.

The ranges for the soil types indicated at the bottom of Fig. 7 are approximate. It is intended that a given design shall be based on actual test results.<sup>13</sup> Thicknesses derived from Fig. 7 are for average conditions based on a tire pressure of 60 lb per sq in., and they should be increased or decreased as much as 20%, depending on tire pressure, thickness and type of pavement and base, characteristics of imported fill, ground-water and drainage conditions, possible effects of frost action, and frequency of loading. In determining the required thickness of pavement and base for the total thicknesses of less than 6 in., the minimum CBR will be used instead of the CBR at 0.1-in. penetration.

<sup>13</sup> "The Preparation of Subgrades," by O. J. Porter, *Proceedings, Highway Research Board, National Research Council, December, 1938, Pt. II, p. 324.*

## ACCEPTED PROCEDURE FOR THE CBR TEST

BY WILLIAM H. JERVIS,<sup>14</sup> M. ASCE, AND JOSEPH B. EUSTIS,<sup>15</sup> ASSOC. M. ASCE

From time to time after the adoption of the California method of design for flexible pavements, the Corps of Engineers found it advisable to modify the method of preparing remolded specimens, so that the CBR test would give results more nearly applicable to field conditions found throughout the United States and in overseas military areas. These modifications were based on the results of a comprehensive laboratory study performed by the Waterways Experiment Station.

The purpose of this paper is to present the pertinent results from the aforementioned investigation and to describe in detail recommended procedures for the preparation of remolded and undisturbed specimens and for the penetration part of the test. However, to give the reader a better understanding of the development of the CBR test by the Corps of Engineers, a brief discussion of the original California method of preparing test specimens and a review of the control tests used by the California Division of Highways and the Corps of Engineers, together with a review of the initial modifications to the California method of preparing test specimens made by the Corps of Engineers, will be presented first.

### CALIFORNIA METHOD OF PREPARING REMOLDED SPECIMENS FOR DESIGN TEST

For the purpose of design, the California State Highway Department developed a mechanical laboratory procedure of preparing the soil for test, as follows:

1. The moisture-density relation for the soil is determined by compacting specimens at several moisture contents in a 6-in.-diameter mold with a static load of 2,000 lb per sq in.
2. For the CBR test, the soil is remolded and compacted at optimum moisture as determined in step 1, under a static load of 2,000 lb per sq in.
3. The test specimen is soaked from the top and bottom for a period of 4 days. During the soaking period the top of the specimen is confined with a surcharge weight of 12½ lb (equivalent to the weight of about 5 in. of pavement).

Information gained from a study of the California procedure indicates that, if because of special conditions it is anticipated that the usual field compaction will not be obtained or that the laboratory method will not duplicate field compaction, a special laboratory procedure is used for preparing the soil samples that will produce a density equivalent to that expected in the field. If the soil is not to be compacted, penetration tests are conducted on undisturbed soaked soils.

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## CALIFORNIA COMPACTION TESTS

For compaction tests, California used the static load method as a standard procedure, both in the laboratory and in the field. In this standard test the moisture-density relation was determined by compacting specimens with a static load of 2,000 lb per sq in. in the CBR mold. However, a dynamic compaction method was used as an alternate field procedure for control on a large percentage of construction jobs, where quick control was required, because of variation in the material.

## COMPACTION CONTROL TEST SPECIFIED BY THE CORPS OF ENGINEERS

The California static compaction test was not considered desirable for construction control because the field laboratories did not have access to the compression machines needed for the tests. The California dynamic compaction test also required special equipment which would have to be constructed. Time was not available to construct equipment and train personnel in the use of this new equipment. The laboratories did have equipment for conducting the compaction test<sup>16</sup> developed by the American Association of State Highway Officials (AASHO); therefore, a modification of this test was developed for construction control. This modification consisted of increasing the weight of the hammer from 5 lb to 10 lb, the height of drop from 12 in. to 18 in., and the number of layers in which the soil was placed in the mold from 3 to 5, and required changing the hammer used in the AASHO compaction test but did not change the molds or other apparatus. A comparison of compactive efforts from limited data available indicated that the density results obtained by this modification of the AASHO compaction test closely approximated those obtained by the California dynamic compaction tests.

EARLY MODIFICATIONS TO CALIFORNIA METHOD OF PREPARING  
REMOLDED SPECIMENS FOR DESIGN TEST

The modified compaction test first appeared as a construction control in the June, 1942, publication of the *Engineering Manual for War Department Construction*.<sup>17</sup> It has since become known as the modified AASHO compaction test. A partly modified California procedure for preparing test specimens was included in this manual. The procedure included the use of the modified AASHO compaction test as a control, and required that CBR test specimens be prepared under a static load of 2,000 lb per sq in. at an optimum water content predetermined by the modified AASHO compaction test.<sup>17</sup>

After a short period of compacting the samples for CBR design tests in accordance with this procedure, it was evident that:

1. The densities of the samples being prepared for the CBR test were at considerable variance with those obtained by field equipment. This variation was particularly true for sandy materials of low plasticity, for which field

<sup>16</sup> "Standard Specifications for Highway Materials," AASHO, Washington, D. C., 1947, test No. T99-38.

<sup>17</sup> "Airfield Pavement Design," *Engineering Manual for War Department Construction*, Office of Chf. of Engrs., U. S. Army, Washington, D. C., June, 1942, Pt. II, Chapter XX.

densities were greater than could be obtained by the static laboratory compaction method.

2. This differential in density between the laboratory and the field on soils of low plasticity gave CBR values which were less than could be expected in the field. Therefore, unless the laboratory density on soils of low plasticity was improved, the CBR values for plastic soils and soils of low plasticity would not be comparable.

In view of the shortcomings of the partly modified California method of preparing test specimens, it was considered necessary to modify the method further so that, when penetrated, the specimens would more nearly reflect the relative stability of the soil as compacted during construction and as later affected by moisture changes. Two methods of compacting the soil for CBR design tests were therefore considered:

- a. A static load method using a variable load to produce the density desired; and
- b. An impact method similar to that adopted for field compaction control tests (modified AASHTO test).

After preliminary study, method *b* was selected to compact the samples for the CBR design test, since (1) the method was similar to that adopted for compaction control tests; (2) the equipment for the dynamic method was available; and (3) static loads greatly in excess of 2,000 lb per sq in. were required to compact sandy samples to the density required. As previously stated, laboratories were not supplied with static load equipment of the high capacity required; and, because of the war, such equipment could not be obtained.

The use of the impact method for the preparation of specimens for design tests was adopted as a second modification by the Corps of Engineers in September, 1942. It was assumed at the time that the main factor in the preparation of the specimens for design tests was to produce the required density. Also, at that time, the use of a surcharge during penetration was introduced into the test procedures to make the test more applicable for use with cohesionless materials. During the test the penetration surcharge was specified to be equal to the anticipated overburden in the field. For cohesionless soils it was specified that a drainage period of 15 min should be allowed for removing any free water remaining on the surface of the specimen after soaking, thus preventing any surface disturbance or softening that would result in low CBR values.

The California Division of Highways, as well as several other state highway departments and agencies, found through experience and field observations that subgrade and base course soils (except clean sands) under impervious pavements usually increased in water content by capillarity and condensation of moisture, regardless of the ground-water elevation, and in some cases became saturated. Although it was recognized that this condition would not occur below all pavements, it was impossible to predict the ultimate moisture accurately. Therefore, the Corps of Engineers, to be conservative, specified a design based on moisture conditions comparable to those obtained in soaked laboratory specimens which were considered to approach the field saturated

condition. For this reason it was stipulated that the CBR design penetration test should be performed on soaked specimens. However, provisions were made to reduce the total pavement thickness 20% for ideal subgrade conditions and for subgrades with low moisture.

#### COMPREHENSIVE LABORATORY STUDIES ON THE CBR TEST

Further modification and refinement in the procedure for the preparation of test specimens and in the method of conducting the CBR test appeared desirable, because of variations in test results obtained between different laboratories on the same material and between successive tests on the same material in one laboratory. Therefore, a program of investigation was formulated in November, 1942, to be conducted at the Waterways Experiment Station. The program included laboratory tests on twenty soils, to determine the effect of different variables on the CBR, such as the method of compaction, water content, density, time of soaking, soaking surcharge, drainage time after soaking, penetration surcharge, rate of penetration, and effect of oversize particles. The principal conclusions and recommendations obtained from the laboratory study are given in the following paragraphs. Details of the study have been published by the Waterways Experiment Station.<sup>4</sup>

The laboratory investigation showed that variations in CBR test results are largely explained by the method of preparing the test specimens. The variations are systematic, however, and are caused primarily by the effects of three variables—molding water content, density, and method of compaction. The variations are probably valid qualitatively; but they may not be strictly valid quantitatively, partly because of the confining effect of the 6-in.-diameter CBR mold. Consistent laboratory results can be obtained only when the variables listed are given full consideration. Unconfined compression tests and triaxial shear tests indicated that the shearing resistances at low strains show trends similar to those found in the CBR test for comparable test specimens. Inasmuch as the molding water content appears to be a prime factor in controlling the structural properties of all except free-draining soils, it follows that, in remolded soil, duplicate laboratory specimens cannot be prepared unless molding water contents are duplicated, even if the water contents and densities obtained subsequent to molding are duplicated. In other words, if a soil is molded at a given water content and then this water content is allowed to increase or decrease by a given amount, another identical soil specimen cannot be reproduced unless the entire cycle is reproduced, starting at the same molding water content.<sup>4</sup> It was felt that dynamic compaction rather than static laboratory compaction produces a soil structure which more nearly simulates that obtained in the prototype by the usual construction methods. This premise was based principally on logic rather than on actual compaction data, since little information was available as to the soil structure produced by field compaction equipment. Subsequently, some field work was accomplished which indicated that, in general, the structure produced by field compaction is more nearly duplicated by laboratory dynamic compaction than by static compaction, as evidenced by unconfined compression tests and by triaxial shear tests.

*Conclusions.*—As a result of the laboratory studies, the following conclusions with respect to the preparation of remolded specimens appear warranted:

1. The maximum dry densities obtained by the modified AASHO dynamic compaction test do not agree with those obtained by the California static method with 2,000 lb per sq in. The variation may be as much as 20 lb per cu ft, depending on the type of soil.

2. Under laboratory dynamic (dropping hammer) compaction, practically all soils develop compaction curves with definite optimum water contents. The compaction curves for soils of low plasticity developed by the static method of compaction often do not show a definite optimum water content.

3. Material should not be used more than once in establishing the water content-density relationship or physical characteristics of the original soil. All control compaction for CBR tests should be performed in the 6-in.-diameter CBR mold with the mold placed on a concrete floor or pedestal for firm support.

4. Small changes in density greatly affect the CBR, especially at high densities in the order of magnitude of modified AASHO or higher. The effect of density is most pronounced for cohesionless soils and soils of low plasticity.

5. Small changes in molding water content greatly affect the CBR of unsoaked laboratory samples, except clean sands and gravels. At a constant density in the unsoaked condition, the higher the molding water content the lower the CBR.

6. The CBR for all soils compacted statically, and soaked, showed an increase with increase in molding water content at a constant density.

7. The CBR for impervious, high-swelling soils compacted dynamically and soaked showed an increase with increase in molding water content for a constant density.

8. The CBR for specimens of soils of low plasticity with little or no swell, when compacted dynamically and soaked, showed an appreciable decrease with a slight increase in molding water content for a constant density. This result is especially significant, since it is generally believed that the shearing resistance is not sensitive to the molding moisture within normal laboratory control for soaked specimens compacted to a given density.

9. Specimens compacted statically, on the dry side of optimum, swell more when soaked than specimens compacted dynamically to the same density at the same moisture.

10. Laboratory soaked specimens of soils with low plasticity compacted at optimum moisture content or compacted at drier than optimum moisture content by the dynamic method have greater CBR values than corresponding specimens compacted by the static method. On one sand-clay tested, the CBR for dynamically compacted soaked specimens was approximately two and one-half times greater than statically compacted soaked specimens at equal densities and at approximately modified AASHO optimum.

11. With one exception, the same trends described for the CBR test in conclusions 6 through 10 also occur in unconfined compression tests and quick triaxial tests when maximum strength and stresses at low strain are considered. The exception is the test involving maximum strength values, with soaked

specimens compacted dynamically, which showed trends similar to those obtained for soaked specimens compacted statically. This agreement is considered to be qualitative only, thus indicating the effect of mold size in CBR tests at high density.

12. The CBR specimen height should not be less than  $4\frac{1}{2}$  in. for all soils.

13. The most practical method for soaking test specimens is by submergence (soaking from top and bottom) for 4 days. Less time is required for some cohesionless soils.

14. In general, the effect of varying the soaking surcharge on the CBR increases with increase in plasticity of the soil. The soaking surcharge greatly affects the CBR of soils with from medium to high plasticity; it moderately affects that of soils of low plasticity; and it has practically no effect on the CBR of cohesionless soils.

15. A satisfactory drainage time for all soils is 15 min.

16. The increase in CBR values for some soils above standard AASHO density may be partly caused by the confining effect of the 6-in.-diameter mold.

Conclusions with respect to the penetration part of the test are as follows:

(1) The penetration surcharge greatly affects the CBR of cohesionless soil, moderately affects that of soils of low plasticity, and has practically no effect on that of soils of from medium to high plasticity.

(2) A penetration rate of 0.05 in. per min will give satisfactory results for all soils.

(3) For consistency and more uniform results, all stress-penetration curves that are concaved upward should be corrected by adjusting the zero point of the curve to establish a new origin.<sup>4</sup>

(4) A closely-controlled constant-strain type of loading apparatus should be used.

(5) Test results on samples containing stones are erratic and modification to the test procedure is needed for these soils. Until such a procedure is developed, several tests should be performed to obtain average representative results.

#### RECOMMENDATIONS

*Specimen Preparation Procedure.*—The equipment required for preparing and testing remolded specimens and undisturbed samples, or for performing field, in-place, CBR tests, is described in detail elsewhere.<sup>18</sup> All soils prepared for the design test should have the same density and moisture conditions as expected in the field. Whenever the modified AASHO method of control compaction is specified, the compaction tests should be performed in the 6-in.-diameter CBR mold using 55 blows per layer. The mold should be placed on a concrete floor or a pedestal and material should not be reused. The following procedures for various types of soil are applicable to specifications of the Corps of Engineers. Two methods of tests are described.

<sup>18</sup> "The California Bearing Ratio Test As Applied to the Design of Flexible Pavements for Airports," *Technical Memorandum No. 219-1*, U. S. Waterways Experiment Station, Vicksburg, Miss., 1 July 1945, p. 30, Figs. 4, 5, and 6, and p. 40, Fig. 9.

**Method 1.**—When no unusual construction or weather conditions exist, method 1 will furnish satisfactory CBR information on plastic soils exhibiting little or no swell. The recommended step-by-step procedure for preparing remolded specimens for test is as follows (the procedure being extended to cover soaking, where testing under this condition is required):

(a) All material larger than  $\frac{3}{4}$  in. should be removed and replaced with an equal proportion of material between 0.18 in. (No. 4 sieve) and  $\frac{3}{4}$  in. in size.

(b) Control compaction tests should be conducted with a sufficient number of test specimens to establish, definitely, the optimum water content for 100% of modified AASHTO density. Four or five specimens should be compacted with water contents within  $\pm 2\%$  of optimum water content, so that the optimum condition can be established rigidly. The fall of the hammer must be controlled carefully and the blows must be distributed uniformly over the specimen. This procedure establishes the moisture content at which specimens for CBR tests should be molded.

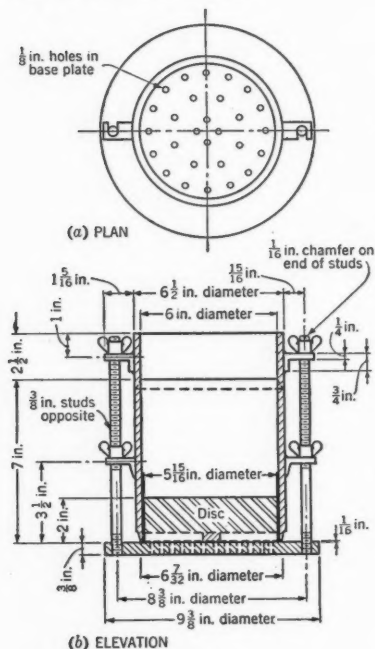


FIG. 8.—VIEW OF STEEL CYLINDER WITH COLLAR

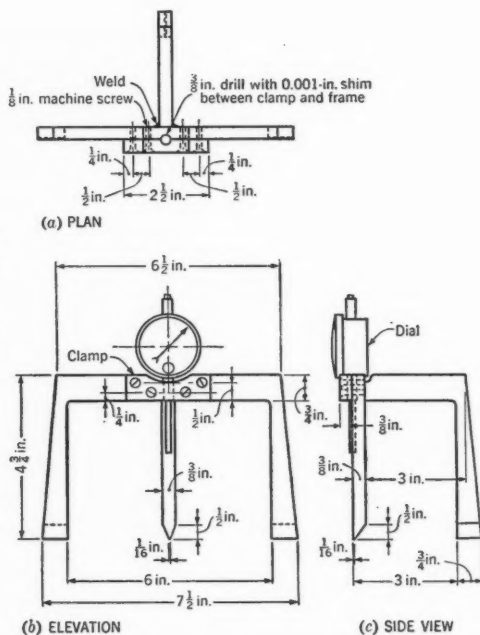


FIG. 9.—VIEW OF STEEL TRIPOD FOR DETERMINING EXPANSION

(c) For the CBR tests the mold should be fitted with an extension collar and base plate (Fig. 8).<sup>19</sup> The mold should be clamped with the fitted extension collar to the base plate and the spacer disk should be inserted over the base plate. A 6-in.-diameter coarse filter paper or wire mesh should be placed on top of the disk.

<sup>19</sup> "The California Bearing Ratio Test As Applied to the Design of Flexible Pavements for Airports," Technical Memorandum No. 213-1, U. S. Waterways Experiment Station, Vicksburg, Miss., 1 July 1945, p. 30, Fig. 5.

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(d) When results are required for a soil at 95% modified AASHO density, three specimens should be compacted at the optimum water content for 100% of modified AASHO compaction, using a different number of blows for each specimen—namely, 55, 25, and 10 blows per layer. The maximum allowable variation in the molding water content should not be more than  $\pm 0.5\%$ . Any specimens not falling within this range should be discarded and a new specimen compacted that does meet this requirement. If specifications call for other than 95% AASHO density or other than 100% AASHO moisture, revisions must be made in this procedure to obtain specimens at the required density.

(e) The collar should be removed, the specimen trimmed, a screen or 6-in.-diameter coarse filter paper placed over the top of the specimen, and a perforated base plate clamped to the top of the test mold.

(f) The test mold should be inverted, the base plate and the spacer disk removed, and the density of the specimen determined.

(g) The adjustable stem and plate should be placed on the surface of the specimen and an annular weight applied to produce an intensity of loading equal to the weight of the base material and the pavement within  $\pm 5$  lb—but the weight shall not be less than 10 lb.

(h) The mold and the weights should be immersed in water to allow free access of the water to the top and the bottom of the test specimen. Initial measurements should be taken for swell, using the dial gage and tripod (Fig. 9).<sup>19</sup> Specimens must be allowed to soak for 4 days. A shorter period of time may be permissible for more pervious materials. Final swell measurements should be taken at the end of the soaking period and the swell computed in percentage of initial specimen height.

(i) The specimen must be taken out of water and free surface water removed, taking care not to disturb the surface of the specimen. The specimen must be permitted to drain downward for 15 min. When removing surface water from impervious samples, it is necessary to tilt the samples. When this is done, the weight should be held firmly in place. The perforated plate and surcharge weights should then be removed and the specimen weighed. The specimen is now considered ready for the penetration test.

(j) The results of tests on all specimens should be plotted to show the relation between density and CBR. The use of this plot can be seen from Fig. 10.<sup>20</sup>

To obtain this plot, specimens were prepared with modified AASHO effort to determine the compaction curve for this effort (shown with points indicated as circles). This series of tests determines the moisture content at which CBR tests should be made (which is optimum for the modified AASHO effort). Two more specimens are then prepared for test at the optimum water content for modified effort using lower efforts of 26 blows and 10 blows per layer.

Three tests have now been made at the optimum moisture for modified AASHO effort, with this effort and two lower efforts. When these specimens have been soaked and penetrated they yield a curve, such as shown in Fig. 10,

<sup>20</sup> "The California Bearing Ratio Test As Applied to the Design of Flexible Pavements for Airports," Technical Memorandum No. 213-1, U. S. Waterways Experiment Station, Vicksburg, Miss., 1 July 1945, p. 38, Fig. 7.

of CBR versus dry density at constant molding moisture content. As will be shown later under method 2, this represents one curve (8% moisture) in the center plot in Fig. 11. This curve intersects the 95% density intercept at a CBR of 45% which is the value which should be used in design.

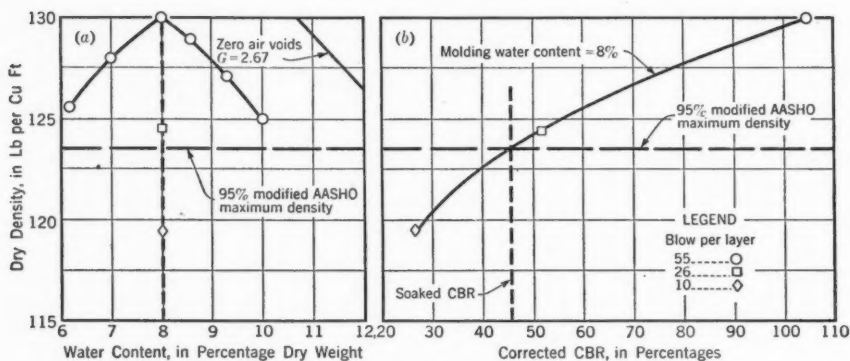


FIG. 10.—METHOD 1; LABORATORY TEST PROCEDURE RECOMMENDED WHEN LACK OF TIME OR EQUIPMENT PREVENTS A MORE THOROUGH INVESTIGATION

It is emphasized that method 1 will give satisfactory results in the laboratory only when the molding water content is controlled closely within the tolerance specified in recommendation (d). Since the tests are performed for one molding water content, no indication (either quantitative or qualitative) is given as to how this soil group will behave if placed in the field at any water content other than that for which the laboratory tests are performed.

**Method 2.**—Where unusual construction and weather conditions are likely to occur and where soils are encountered that are very critical with changes in moisture, method 2, as described herewith, is recommended for plastic soils exhibiting little or no swell, as it furnishes more complete information. CBR test results are affected by the density and molding water content of the soil specimens. If detailed information is desired regarding the effect of molding water content and density on CBR over a wide range of values, it is recommended that the variations be determined for typical soils encountered on the job. A series of specimens should be prepared and tested, as follows:

(a) All specimens should be prepared in a manner similar to that outlined under method 1, except that each specimen used in the development of the 55-blow compaction curves should be penetrated. In addition, the complete compaction curves for the 25-blow and 10-blow per layer compactive efforts should be developed and each test specimen compacted should be penetrated. If both soaked and unsoaked data are desired, the specimen may be molded with alternate samples being soaked before penetration. All compaction is performed in the 6-in.-diameter CBR mold, using the 10-lb hammer dropped 18 in. on each layer. It may be necessary to include an effort greater than the 55-blow effort, in the event greater compaction in the field is possible, as a result of initial rolling or traffic with very heavy wheel loads.

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(b) The data from method 2(a) should be plotted as shown in Fig. 11.<sup>21</sup> As in method 1, the material was a clay-sand with a liquid limit  $w_L = 17$  and a plasticity index  $I_P = 4$ . It is assumed that specified field control required a minimum dry density of the 95% modified AASHO maximum and a water content range of from 6% to 8%. All specimens were packed in five layers in the CBR mold using the 10-lb hammer and an 18-in. drop, after which they were soaked for 4 days in the CBR mold and then penetrated. As in method 1, a surcharge of 10 lb was applied for the soaking and the penetration tests. The tests plotted in Fig. 11(b) revealed no swelling during the time that the specimen was being soaked. The shaded area in this illustration is the region that will yield a CBR range of from 46% to 105%.

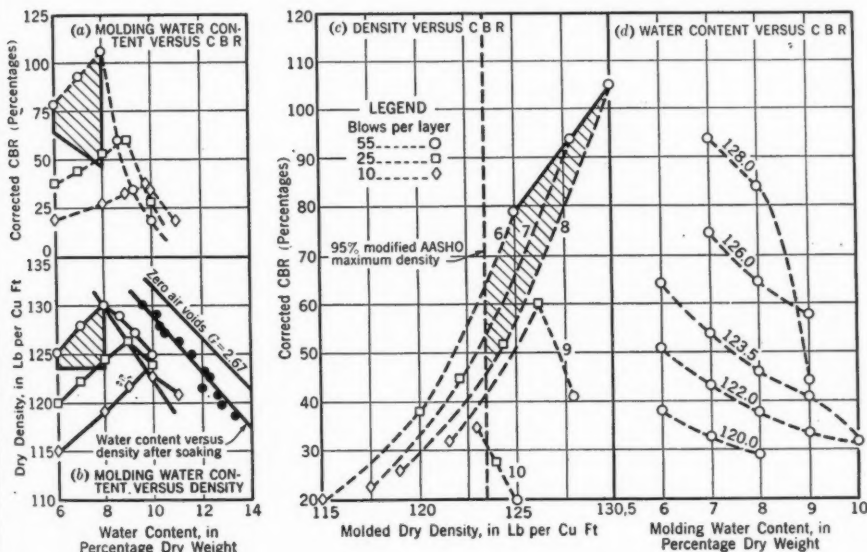


FIG. 11.—METHOD 2; LABORATORY TEST PROCEDURE RECOMMENDED WHEN A THOROUGH INVESTIGATION IS DESIRED

Method 2 is valuable to obtain test results on soils that are greatly affected by small changes in density and molding water content; it supplies a picture of the CBR characteristics, within the range of the field control expected, which will be useful in establishing the limiting CBR values. The test results as obtained by method 2 should be used in connection with the design curves, with the full understanding that the variations obtained may be valid only qualitatively.

*Comment.*—The procedure for preparing specimens of all swelling soils is the same as that for soils of low plasticity, except that the specimens should be prepared at a water content and a density controlled by swelling tests. The swelling tests are performed on the samples used in developing the compaction

<sup>21</sup>"The California Bearing Ratio Test As Applied to the Design of Flexible Pavements for Airports," Technical Memorandum No. 213-1, U. S. Waterways Experiment Station, Vicksburg, Miss., 1 July 1945, p. 39, Fig. 8.

curves. Method 2 is the most desirable in this case, in that full cognizance can be taken of the swelling effects for three compactive efforts. On the basis of the density and the moisture conditions to be expected in the field, the proper CBR value can be selected from the family of curves. If method 1 is used, the moisture content at which swelling ceases is noted. This cessation usually occurs at optimum or on the wet side of optimum. Specimens for the CBR test are then prepared at the moisture and the density indicated. If the moisture control at the point of swell is appreciably on the wet side of modified AASHO optimum a compactive effort (less than modified) should be chosen for which the selected moisture is approximately optimum.

Cohesionless soils ( $I_P$  less than 2) that will readily compact under rollers or traffic, to the maximum density specified by the modified AASHO method, should be prepared as described in method 1, except that specimens should be prepared at 100% modified AASHO maximum density only for the CBR penetration test. Several specimens should be tested for CBR and the average used for design. Cohesionless soils that do not compact readily should be prepared as described in method 1 at 95% of modified AASHO maximum density. In general, soaking will not lower the CBR of cohesionless soils and may be omitted. However, on occasional cohesionless soils, saturation may be a factor, in which case the specimen should be soaked.

The method of soaking undisturbed samples in molds or cylinders is the same as that for remolded samples in the CBR mold described in method 1. After soaking, making swell measurements, allowing for drainage, and making density determination, the samples are considered ready for the penetration test. If this type of sample is not to be soaked, the seal coating is removed from one end of the container, the surface is made level, and the penetration test is performed in the usual manner.

The method of soaking undisturbed samples taken in boxes can be improvised and swell readings can be obtained using the same perforated plate, surcharge weights, and tripods as used with the standard CBR mold. Since the boxes are usually coated before being sent to the field, the swell in the boxes themselves is negligible. In the event soaking is not required, the seal coating can be removed from one end, the surface leveled with a thin cover of sand if necessary, and the penetration test conducted with the standard testing equipment in the usual manner. The fact that the box construction must be rigid should be stressed.

*Penetration Test Procedure.*—The laboratory step-by-step procedure for this phase of the design test is as follows, and should be used on undisturbed or remolded samples after the testing surface has been prepared:

(a) A penetration surcharge should be applied on all soils sufficient to produce an intensity of loading equal to the weight of the base material and the pavement (within  $\pm 5$  lb)—if a pavement is to be constructed which will overlie the soil in the prototype represented by the sample, except that the weight shall not be less than 10 lb. The weight applied must be estimated; and, if it does not produce the intensity present in the final design, the test should be repeated. If the sample has been previously soaked, the penetration

surcharge should be equal to the soaking surcharge which, in turn, should have been estimated and governed by the conditions described in method 1(g). On soils in a condition for which it is expected that a low CBR value will be obtained, it is advisable to apply the penetration piston and the penetration surcharge weights in either one of two ways, to prevent upheaval of the soil into the hole of the surcharge weights. In the first method, one 5-lb annular disk surcharge should be applied to the soil surface, the penetration piston then seated with a 10-lb load, and finally the remainder of the surcharge applied by the use of slotted 5-lb surcharge weights. In the alternate method, a special locking and alinement device as shown in Fig. 12 can be used.<sup>22</sup> (The  $\frac{1}{4}$ -in. pin at the top is so fitted that the penetration piston is flush with the bottom of the surcharge weight.)

(b) The penetration piston should be seated with a 10-lb load and the dial gage set at zero. The purpose of a 10-lb load before starting the penetration test is to insure satisfactory seating of the piston, and it should be considered as the zero load when determining stress-strain relations.

(c) Load on the penetration piston should be applied so that the rate of penetration is approximately 0.05 in. per min. Load readings should be obtained at deformations of 0.025 in., 0.050 in., 0.075 in., 0.1 in., 0.2 in., 0.3 in., 0.4 in., and 0.5 in. In using manually operated loading devices, it may be necessary to take more load readings as an assistance in controlling the rate of penetration.

(d) The moisture content should be determined in the upper inch and, in the case of the laboratory tests, for the entire depth of the sample.

(e) The penetration load in pounds per square inch should be computed and the stress-penetration curve drawn. To obtain true penetration loads from the test data, the zero point of the curve should be adjusted to correct for the initial concave-upward shape, if present.

(f) The corrected load value should be determined at 0.1-in. and 0.2-in. penetration. Next, the corrected CBR should be determined for 0.1-in. and 0.2-in. penetration by dividing the load at 0.1 in. and 0.2 in. by the standard loads of 1,000 lb per sq in. and 1,500 lb per sq in., respectively. Each ratio should be multiplied by 100 to convert to percentages.

(g) The CBR usually selected is at 0.1 in. penetration. If the CBR at 0.2-in. penetration is greater than that at 0.1-in. penetration, the test should

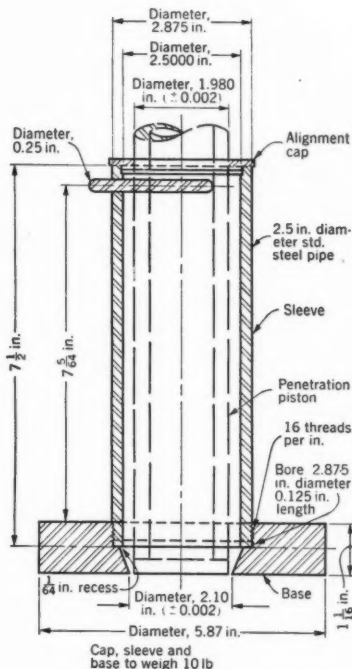


FIG. 12.—LOCKING AND ALINEMENT DEVICE FOR PENETRATING SURCHARGE WEIGHTS AND PISTON

<sup>22</sup> "The California Bearing Ratio Test As Applied to the Design of Flexible Pavements for Airports," Technical Memorandum No. 213-1, U. S. Waterways Experiment Station, Vicksburg, Miss., 1 July 1945, p. 31, Fig. 6.

be rerun. If check tests give similar results, the CBR at 0.2-in penetration should be used.

The step-by-step procedure recommended in the conduct of field in-place CBR tests is as follows:

a. The surface to be tested should be prepared by removing loose and dried material, leveling the test area as perfectly as practicable.

b. The truck<sup>23</sup> should be located so that the center of the iron beam on the rear end is directly over the surface to be tested. The track jacks should be placed beneath the ends of the iron beam and the truck lifted so that no weight rests on the springs and so that the truck is approximately level. The truck should be loaded sufficiently to give a counterreaction of about 7,000 lb.

c. The swivel head and test jack should be installed at the underside and the center of the iron beam, the proving ring connected to the end of the jack, and the penetration piston placed to from within 1 in. to 2 in. of the surface to be tested. The rod level should be fastened to the pipe extension and the swivel head adjusted until the penetration piston is plumb. Then the apparatus should be locked in position by tightening the clamping nut in the swivel head.

d. The 10-in. steel surcharge plate should be placed beneath the penetration piston so that when the piston is lowered it will pass through the center hole.

e. The penetration piston should be seated under a 10-lb load (for rapid setting use high-gear ratio of jack).

f. The surcharge plate should be raised while the seating load is on the piston and clean fine sand spread to a depth of from  $\frac{1}{4}$  in. to  $\frac{3}{8}$  in. over the surface to be covered by the plate. This procedure serves to distribute the weight of the surcharge uniformly.

g. Surcharge weights should be applied to the steel plate equivalent to the load intensity of the material and to the pavement which will overlie the subgrade or the base, except that the minimum weight applied should be made up of the 10-lb circular steel plate plus one 20-lb surcharge weight. This minimum weight creates an intensity of loading equal to that created by the 10-lb weight used in the 6-in.-diameter CBR mold in the laboratory.

h. The penetration dial clamp should be attached to the piston so that the dial rests on the dial support.

i. The dial gages should be set at zero.

j. Load should be applied to the penetration piston so that the rate of penetration is approximately 0.05 in. per min. By using the low-gear ratio of the jack during the test, a uniform rate of penetration can be maintained by the operator. The deflection of the proving ring should be recorded at penetration depths of 0.025 in., 0.05 in., 0.075 in., 0.1 in., 0.15 in., 0.2 in., 0.3 in., 0.4 in., and 0.5 in. The CBR is then computed as in the laboratory test.

k. At the completion of the test, a sample should be obtained at the point of penetration for water content determination. A sample should also be obtained from about 4 in. to 6 in. away from the point of penetration, for density determination.

<sup>23</sup> "Development of California Bearing Ratio Apparatus," by W. J. Turnbull, *Civil Engineering*, May, 1945, p. 234.

## TEST SECTION NO. 1, STOCKTON FIELD, CALIFORNIA

BY O. J. PORTER,<sup>5</sup> M. ASCE

One of the major field tests in the research program instigated to check the tentative design curves was the accelerated traffic test conducted at Stockton Field in 1942. Authorized by the Office of the Chief of Engineers, the work was done by personnel of the California Division of Highways and of the Sacramento District of the Corps of Engineers, under the supervision of the writer.

### DESCRIPTION OF PROJECT

The test section was located at Stockton Field for a number of reasons. In the first place, runways and taxiways that had been constructed in 1936 over the adobe subgrade (CL-OL in the Airfield Classification,<sup>7</sup> A-6 and A-7 in the PRA classification<sup>6</sup>) had failed during the winter of 1940-1941 under the operation of AT6A training planes having wheel loads of only 2,000 lb. This performance indicated that the subgrade was of about as poor a quality as could be expected at any proposed airport site; hence, tests at this site would be valuable in checking the tentative design curves in the low CBR range.

In the second place, the moisture content of the subgrade and the subsoil had become uniformly equalized under an old paved taxiway that had not been subjected to traffic and therefore had not failed. This taxiway provided a satisfactory area for the test section and also for determining the magnitude of the pavement deflection caused by the action of the AT6A planes (2,000-lb wheel load) that caused failure of the adjacent runways.

Before constructing the test section, borings were made to a depth of 8 ft beneath both the taxiway and the old runway to determine the character and the uniformity of the subgrade and the subsoil. Undisturbed samples were also obtained under the selected area to determine the moisture content and the bearing ratio of the subgrade. These explorations indicated that the old runways and taxiways consisted of about 6 in. of sandy loam soil, the top 3 in. or 4 in. being stabilized and then sealed with emulsified asphalt. Both the untreated sandy loam and the stabilized material appeared well compacted. The stabilized material also appeared relatively dry and very stable, although somewhat brittle. The subgrade and the subsoil to a depth of 3 ft consisted of black adobe of CL-OL classification (A-7 in the PRA classification). This material contained approximately 30% moisture, and laboratory tests on undisturbed samples gave CBR values ranging from 4% to 8% at 0.1-in. deflection and averaging approximately 5%. Remolded samples compacted to the field density at the natural water content gave CBR values of 2% when confined by a surcharge of 0.45 lb per sq in. during the soaking period and of 8% when confined by a surcharge of 2 lb per sq in. (the approximate pressure beneath 2 ft of pavement and base) during the soaking period.

The subsoil below the black adobe to a depth of 8 ft was found to be a relatively compact gray clay of CL-OL classification (A-7 in the PRA classifica-

tion) containing from 25% to 30% moisture. In general, the soil conditions were similar beneath the runways that failed and the taxiway area selected for the runway test section, which did not fail.

The runway test section (Fig. 13) was constructed on top of the old taxiway after measurements had been made of the deflection produced by both static and dynamic wheel loads of the AT6A plane. Some of the deflection points were left in place to permit measurement of the deflection of the bottom of the

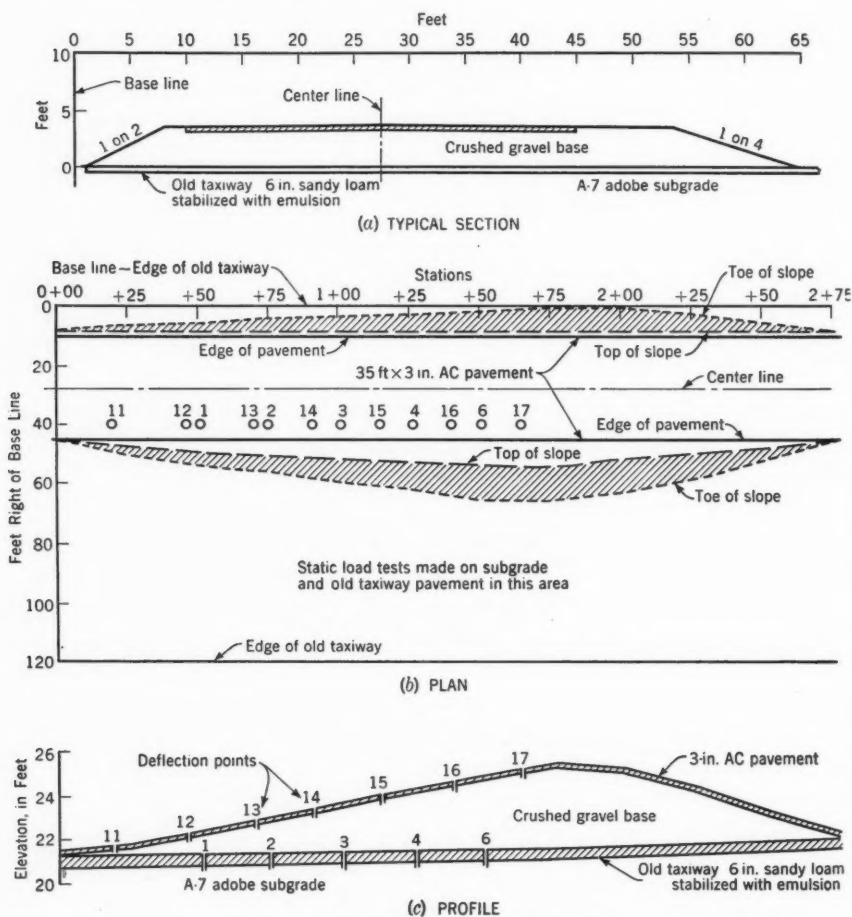


FIG. 13.—STOCKTON RUNWAY TEST SECTION

base beneath the completed test section. The old pavement was scarified lightly and then rolled before placing new material to provide a bond and to eliminate any possibility of the base slipping on the hard smooth surface of the old taxiway. With the exception of the thickness of sandy loam on the old taxiway, the entire base course consisted of a well-graded mixture of sand and gravel, and of crushed gravel. Whenever the total thickness of pavement

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and base is referred to in this paper, it includes the 6-in. thickness of sandy loam on the old taxiway.

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The base material was obtained from a commercial plant at Tracy, Calif., and the aggregate gradation was closely controlled to insure uniformity throughout the length and the depth of the base on the test section. This material is approximately comparable in quality to many of the pit-run and bank-run gravel bases readily available in many areas of the western states. After soaking, this material had a CBR of 85%. The base was thoroughly compacted in layers from 4 in. to 5 in. thick, placed at optimum moisture to a relative compaction of from 87% to 100% of the maximum density obtained in the static compaction test of the California Division of Highways. Although each layer was rolled and compacted with a 12-ton roller, a small degree of consolidation occurred during the traffic tests as the result of the repeated action of the heavy wheel loads. Where the thickness of the base course ranged from 3 ft to 4 ft, settlements from  $\frac{1}{2}$  in. to  $\frac{3}{4}$  in. occurred during the traffic test without causing failure of the pavement. This action is consistent with highway experience where repetitions of dynamic wheel loads produce a greater density than is obtainable during construction. Even when the material is placed in thin layers, the moisture content is carefully controlled, and heavy rollers are used.

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Asphaltic concrete pavement 3 in. in compacted thickness was placed over the entire test section. The pavement mixture contained approximately 5% material passing a 200-mesh sieve and 5.4% of 70 penetration asphalt (by weight) to provide a highly flexible pavement without inducing instability in the mixture. The pavement remained stable and showed no signs of either flushing or displacing during the traffic tests. It also proved very flexible and withstood the repetitions of the wheel loads, satisfactorily and without cracking, where the thickness of the base was sufficient to eliminate plastic flow of the subgrade. In areas having insufficient thickness, substantial grooving developed before serious cracking and initial failure occurred.

#### TESTING PROGRAM

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The objective of the test section was to check or to modify the tentative design curves by the actual service behavior of a test pavement constructed on a subgrade with a low CBR value. Information was desired for wheel loads ranging from a few thousand pounds to approximately 53,300 lb. Because of lack of time, it was impossible to make complete traffic tests with large numbers of repetitions of wheel loads over the full range investigated; hence, it was desired that indirect observations be used to the fullest possible extent to predict the behavior of the test section. Therefore, direct traffic tests were made under wheel loads of 25,000 lb and 40,000 lb, and the deflections of the pavement and of the subgrade were measured under both static and moving wheel loads of 2,000 lb, 5,000 lb, 10,000 lb, 25,000 lb, 40,000 lb, and 53,300 lb. Static bearing tests on circular steel plates were also made to provide a basis for possible correlation with measurements under pneumatic wheel loads used in the traffic tests and in the deflection observations.

*Pavement Deflection.*—Deflection measurements under both static and moving wheel loads had been used rather extensively by the California Division of

Highways in its study of the performance of flexible pavements. Special electrical equipment developed by that division permitted the measurement of very small deflections under rapidly moving wheel loads. Highway experience indicated that the deflection of flexible pavements under the maximum highway wheel load of 10,000 lb should not exceed 0.02 in. or 0.03 in. in cases where a large number of repetitions (a million or more) is anticipated. Because of the lesser number of repetitions anticipated on airfield pavements, deflections of approximately 0.05 in. were judged permissible provided that the pavement is good and does not become brittle. Also, the allowable deflections under the heavier wheel loads should be slightly larger because of the larger contact areas of the heavier loads which produce a flatter curvature of the pavement.

Gages for the measurement of deflection (vertical movement) were placed at five points on the surface of the subgrade and at seven points on the surface of the pavement (for positions, see Fig. 13(c)). The gages were installed at locations selected to cover the range between 12 in. and 48 in., the total thickness of pavement and base, and were placed 5 ft inside the right edge of the pavement. The deflection measurements for each wheel load were first made at the point having the least thickness and then progressed up the ramp to the point of maximum thickness. This order was followed to reduce the effect of load repetition to a minimum. The deflection tests were made with the following wheel loads in the order listed:

Wheel loads, in pounds	Equipment used
2,000.....	AT6A airplane
5,000.....	Dump truck
10,000.....	RU LeTourneau Carryall
25,000.....	RU LeTourneau Carryall
25,000.....	Bomber B-24-D
40,700.....	RU LeTourneau Carryall
53,300.....	RU LeTourneau Carryall

The deflection of the pavement under dynamic wheel loads was measured at a speed of about 10 miles per hr. For each wheel load dynamic tests were made prior to the static tests. In the static tests, the total deflection was recorded after the wheel load had remained over the deflection point until movement had apparently ceased. The time varied from 3 min to 7 min.

A summary of deflection measurements under the foregoing wheel loads is presented in Table 1. Points of interest relative to each of the wheel loads are reviewed subsequently.

*2,000-Lb Wheel Load.*—Deflections were observed at five locations on the old taxiway pavement (6 in. thick), under a wheel load of 2,000 lb. The average results of the five readings were as follows:

Loading condition	Deflection, in inches
Dynamic.....	0.079
Static.....	0.092

All these areas contained cracks; hence, an extra test was made in a location free of cracks. This test resulted in a dynamic deflection of 0.034 in. and a

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static deflection of 0.048 in., which are both less than half the values observed for the cracked areas. Fine hair cracks developed while the plane was taxiing over the test area. This behavior clearly indicated that the pavement was not capable of withstanding deflections of approximately 0.05 in. for even a few repetitions. Additional repetitions would have been required, of course, to increase the number and the size of the cracks and to cause complete failure.

TABLE 1.—PAVEMENT AND SUBGRADE DEFLECTION, IN INCHES,  
STOCKTON (CALIF.) TEST SECTION No. 1

Pres- sure <sup>a</sup>	Wheel load, in kips <sup>b</sup>	3-IN. AC PAVEMENT							OLD TAXIWAY					
		Deflection Points; see Figs. 13(b) and 13(c):												
		11	12	13	14	15	16	17	1	2	3	4	6	
Thickness <sup>c</sup>		12	18	24	30	36	42	48	20	26	32	38	45	
60	{5.0D.....	0.068	0.045	0.030	0.033	.....	.....	0.023	0.018	0.015	0.012	.....	0.005	
	{5.0S.....	0.096	0.060	0.042	0.045	.....	.....	0.048	0.035	0.019	0.015	.....	0.009	
58	{10.0D....	0.094	0.088	0.074	0.068	0.062	0.050	0.046	0.056	0.036	0.025	0.018	0.013	
	{10.0S....	0.162	0.137	0.082	0.074	0.064	0.056	0.059	0.083	0.046	0.034	0.024	0.018	
58	{25.0D....	0.246	0.210	0.160	0.157	0.134	0.110	0.095	0.143	0.119	0.097	0.075	0.046	
	{25.0S....	0.485	0.345	0.205	0.184	0.160	0.129	0.114	0.240	0.156	0.130	0.094	0.062	
	{25.0D <sup>d</sup> ....	.....	.....	.....	.....	.....	.....	.....	.....	0.093	0.073	.....	.....	
	{25.0S <sup>d</sup> ....	.....	.....	.....	.....	.....	.....	.....	.....	0.114	0.086	.....	.....	
58	{40.0D....	0.550	0.350	0.187	0.180	0.160	0.128	0.110	.....	0.148	0.129	0.095	0.067	
	{40.0S....	..... <sup>e</sup>	..... <sup>e</sup>	0.252	0.237	0.190	0.151	0.146	.....	0.206	0.167	0.120	0.086	
58	{53.3D....	..... <sup>e</sup>	..... <sup>e</sup>	0.261	0.217	0.173	0.164	0.138	..... <sup>e</sup>	0.216	0.158	0.110	0.086	
	{53.3S....	..... <sup>e</sup>	..... <sup>e</sup>	..... <sup>e</sup>	0.272	0.226	0.210	0.176	..... <sup>e</sup>	0.269	0.223	0.162	0.122	

<sup>a</sup> Contact pressure, in pounds per square inch. <sup>b</sup> 1 kip = 1,000 lb or 1 "kilo-pound"; D = dynamic and S = static. <sup>c</sup> Thicknesses of pavement and base in inches. <sup>d</sup> B-24-D bomber. <sup>e</sup> Deflection was too great to be measured properly with electric gages.

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**5,000-Lb Wheel Load.**—Deflections under the 5,000-lb wheel load are given in Table 1. Bearing tests on a rigid plate resulted in pavement deflections from 110% to 150% of the static wheel load values. This relationship is just opposite to that found for the heavier wheel loads.

At this wheel load, tests were made to determine the effect of tire pressure on the deflection of the top of the pavement and on the deflection of the top of the subgrade. These tests indicated that the changes in air pressure in the tires of from 40 lb per sq in. to 80 lb per sq in. greatly influenced the deflection of the top of the pavement, but had little influence on the deflection of the top of the subgrade. Although the results were only sufficient to show a trend, it is logical to assume that the higher tire and contact pressures are of particular importance in selecting pavement and base materials for the upper part of a roadway or a runway.

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**10,000-Lb Wheel Load.**—Bearing tests made on a rigid plate having a contact area comparable to that of the tire used for the 10,000-lb wheel load tests resulted in from about 20% to 25% less deflection than that obtained for the

static wheel load. Referring to Table 1, it may be noted that a deflection of approximately 0.04 in. was obtained at the bottom of the base for a total thickness of 26 in. Highway experience in California indicates that a minimum of 24 in. of flexible pavement and base is desirable over this class of soil on roads that carry a large volume of heavy truck traffic (maximum wheel loads of approximately 9,000 lb). Deflection measurements have also indicated that for highway conditions the dynamic deflections should not exceed 0.03 in. or 0.04 in. and that even this amount is permissible only in the event the flexible pavement is good and not brittle. Observations made during seventy-six trips over the pavement required for this series of tests indicated that some plastic flow of the subgrade may have occurred during the time interval of the static load test for thicknesses less than 25 in. Thus, additional traffic of even a rolling load would probably have caused progressive plastic deformation resulting in grooving and ultimate failure after a sufficient number of load repetitions.

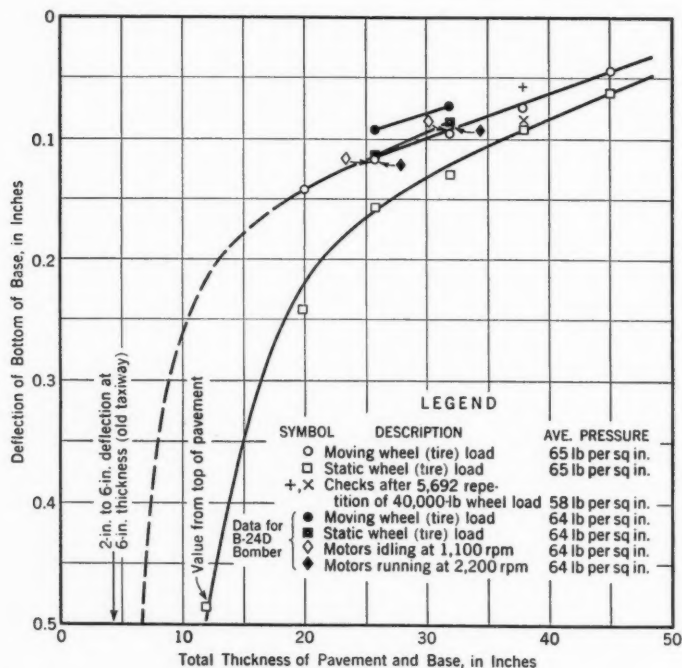


FIG. 14.—DEFLECTION OF THE BOTTOM OF THE BASE UNDER A LOAD OF 25,000 LB

#### 25,000-Lb Wheel Load.—

LeTourneau Carryall.—Dynamic and static deflections measured under the 25,000-lb wheel load are included in Table 1. These data are also shown in Fig. 14, a typical plot of the deflection of the bottom of the base versus total thickness. Bearing tests with a footprint area and a contact pressure corre-

sponding to that of the tire used for the 25,000-lb test resulted in deflections from 35% to 55% less than the values measured with the static wheel load. The deflection was checked at the 36-in. thickness after traffic was completed with the 40,000-lb wheel load to determine the increased deflection that resulted from the cracking and the weakening of the pavement and the base. The pavement at this point showed a 15% increase in deflection after the development of cracking. The deflection of the bottom of the base at the 38-in. thickness, where the cracking had not yet developed to an appreciable extent, was slightly less than readings obtained before traffic. The indications are that no plastic flow occurred at a thickness of 38 in. Any plastic flow of the subgrade probably will cause progressive grooving, cracking, and failure with additional load repetitions. The rate of failure in relation to load repetition would undoubtedly depend to a large extent on the magnitude of the plastic deformation per repetition.

**B-24-D Bomber.**—A B-24-D bomber was made available for a few hours by the Fourth Air Force. In view of the limited time that the bomber was available, deflection tests were made only at the bottom of the base at two points. Dynamic and static deflections measured at those two points are included in Table 1 and Fig. 14. It will be noted that the deflections caused by the bomber were approximately 20% less than those for similar tests made with the carryall having a wheel load of 25,000 lb. An attempt was made to obtain a quick but accurate determination of the wheel load and footprint area for the bomber. Difficulty was encountered in making these measurements; therefore, the accuracy was not as great as desired. As closely as could be determined, the wheel load was 24,000 lb and the contact pressure was 60 lb per sq in. In addition to the deflection measurements for the B-24-D bomber that are included in Table 1, tests were made to determine the effect of the vibration of the motors. These tests (plotted in Fig. 14) indicated that vibrations increased the deflections only slightly over the values obtained for the static load with the motor dead. Although these results are 25% higher than the deflections measured under the rolling wheel load, it is probable that if the tests had been made on a lesser thickness of pavement the influence of the dynamic load would have been much greater.

**40,000-Lb Wheel Load.**—Values of deflection measurements for the 40,000-lb wheel load, given in Table 1, are maxima. The longitudinal distribution of deflections under the moving 40,000-lb wheel load is given in Fig. 15. The data for these curves were obtained from oscillograph films for the deflection of points 3 and 14, Fig. 13. The footprint was 20.25 in. wide by 31.25 in. long, or 568 sq in., and the average tire contact pressure during the tests was 72 lb per sq in. In tests with the 40,000-lb wheel load and the 53,000-lb wheel load, the 24-in. by 32-in. tire was overloaded and the air pressure (59 lb per sq in.) could not be dropped sufficiently to reduce the contact pressure to the desired range of from 60 lb per sq in. to 65 lb per sq in. Heavy bombers, however, usually carry higher air pressure and therefore the deflection curves probably approximate actual field service conditions. Bearing tests on rigid circular plates gave somewhat erratic results but in general indicated from 15% to 45% less deflection than was obtained under the static wheel load. A similar dis-

crepancy has been found in other tests, and it appears that the shape and the rigidity of the bearing plate greatly influence the static load test results. Under actual tire loads the pavement can bend throughout the loaded area, whereas with the rigid plates essentially all the bending of the pavement and the base must occur outside the loaded area.

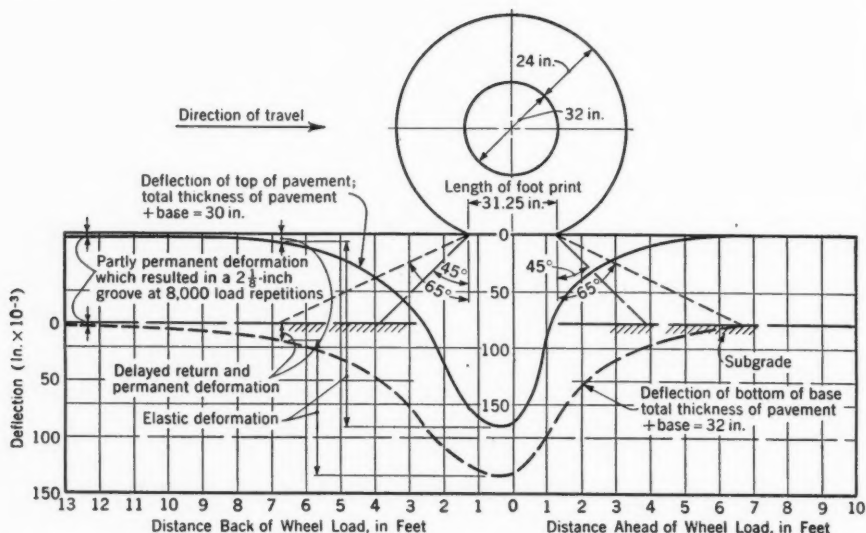


FIG. 15.—TYPICAL CURVES SHOWING THE LONGITUDINAL DISTRIBUTION OF PAVEMENT DEFLECTION; WHEEL LOAD OF 40,000 LB MOVING AT APPROXIMATELY 10 MILES PER HOUR

**53,300-Lb Wheel Load.**—Deflections measured under the 53,300-lb wheel load are included in Table 1. Bearing tests made on the pavement where the total thickness was 18 in. gave very erratic results. Disregarding these tests, the deflections at the other thicknesses varied from 10% to a very large indeterminate amount less than the values obtained for the static wheel load. Large deflections were obtained for the thinner pavement sections. Tests under this heavy load and also under the 40,000-lb load indicate large differences between the deflection at the top of the pavement and at the bottom of the base. The values measured at the top of the pavement were higher primarily because of elastic deflection and, to a lesser degree, because of the consolidation deformation of the base. Even if the thickness of base were entirely adequate to eliminate all plastic deformation of the subgrade, the flexible pavement must be of good quality and very flexible to withstand a large number of repetitions of deflections in the order of the minimum for this load, which was 0.138 in.

#### TRAFFIC TESTS

The left side of the test section was subjected to the traffic of a 25,000-lb wheel load for a maximum of 1,920 coverages after the static load bearing tests had been completed on top of the pavement. Similarly, the right half of the test

section was subjected to 3,730 coverages of a 40,000-lb wheel load. For the first 1,600 repetitions, traffic with the 40,000-lb load was concentrated in one rut, but after 1,600 repetitions traffic was spread over an area about three times the width of the tires. Traffic with the 25,000-lb wheel load was spread over three tire widths for the entire period. Traffic lanes were developed near the edges of the test section for the outer wheels of the equipment and near the center of the section for the inner wheels of the equipment. Since the behavior of the outer lanes was influenced by the proximity of the wheels to the edge of the pavement, only the behavior of the inner lanes is evaluated in the traffic tests. As traffic progressed, measurements were made to determine the amount of grooving and the extent of cracking in the pavement.

Observations were recorded that show the extent of cracking of the pavement, rutting, grooving, and roughness. It is considered that the extent to which these detrimental behavior characteristics progressed from the thin end toward the thick end of the test section is an indication of the extent of shear deformation in the subgrade for the following reasons: The 3-in. asphaltic concrete was flexible and of good quality; therefore, no part of the detrimental behavior can be attributed to weakness of the asphaltic concrete. The base course was of good quality and well compacted; no evidence of shear deformation was found in the base course and only a nominal amount of consolidation occurred. Therefore, the cause of the detrimental behavior must have been in the subgrade. The adobe clay subgrade was practically saturated and was too impervious to drain, which prevented any appreciable compaction or consolidation occurring under traffic. The cracking, rutting, grooving, and roughness that occurred are attributed, therefore, to plastic flow or to shear deformation in the subgrade.

**25,000-Lb Wheel Load.**—Deep rutting and complete failure progressively developed during the 1,920 coverages of the 25,000-lb wheel load from the thin end of the pavement to the 16-in. thickness. Cracking and grooving of the pavement to the extent classified as a failure occurred to approximately the 20-in. thickness. Most of the 1,920 coverages were made in relatively warm weather. Because of this and the very flexible character of the pavement, cracks did not develop at greater thicknesses even though the surface grooved and roughened to the 30-in. thickness.

A 20-in. thickness of pavement and base was inadequate to protect the subgrade for 1,920 coverages. The data indicate that a 23-in. to 28-in. thickness would be required to prevent detrimental shear deformation in the subgrade. Since the subgrade had an average CBR of 5%, these values are in substantial agreement with the 26-in. thickness required by the design curves for a subgrade having a CBR of 5%.

**40,000-Lb Wheel Load.**—Failures developed progressively with traffic repetitions from the thin end to the thicker parts of the test section and ultimately to the point where the total thickness of pavement and base was 30 in. This thickness was not sufficient to prevent excessive shear deformation in the subgrade for 3,900 coverages of the 40,000-lb wheel load. Cracking also occurred to greater thicknesses of pavement and base, but generally disappeared under the action of traffic in warm weather.

A 30-in. thickness of pavement and base was not adequate to protect the subgrade from shear deformation. The data indicate that 32-in. to 36-in. thickness would be required to prevent excessive shear deformation in the subgrade. For a subgrade with a CBR of 5 (which is the average CBR of this subgrade), the design curves call for 32 in. of base and pavement for a 40,000-lb wheel load. The 32-in. to 36-in. range indicated by these tests is only slightly greater than the thickness required by the design curves.

#### CONCLUSIONS

It appears that the following conclusions can be drawn from the traffic and deflection data for soil, base, and pavement conditions comparable to those present on this test project:

- (a) The results obtained from the traffic tests are indicative of the service behavior of runway pavements where traffic is not concentrated near the edge of the pavement.
- (b) The thicknesses of pavement and base required according to the test are in reasonable agreement with those required by the CBR design curves.
- (c) Some increase in the total thickness of pavement and base as shown by the design curves is indicated for taxiway operations where a large volume of traffic may be concentrated near the edges of the pavement.
- (d) The base used in the test section was good and remained stable throughout the traffic tests.

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# SERVICE BEHAVIOR TESTS, BARKSDALE FIELD, SHREVEPORT, LA.

BY RALPH HANSEN,<sup>24</sup> ASSOC. M. ASCE

Another research program conducted to obtain information for the design curves was the flexible pavement investigation conducted at Barksdale Field, near Shreveport, where special test tracks were built to study the base requirements on a plastic clay subgrade. Barksdale Field was selected for the service behavior tests as it was believed that the subgrade at this site would be representative of the plastic clay subgrades which might be encountered in the construction of flexible pavements in the southern part of the United States. The specific purpose of the tests was to determine the following factors for flexible pavements using 20,000-lb and 50,000-lb wheel loads:

- (1) The total thickness of pavement and base necessary to prevent detrimental shear deformation in the subgrade;
- (2) The effect of the quality of base materials on the total specified thickness of pavement and base;
- (3) The quality of base materials required directly under the pavement; and
- (4) The adequacy of the compaction requirements as adopted by the Chief of Engineers<sup>25</sup> to prevent detrimental consolidation of the base materials and the subgrade.

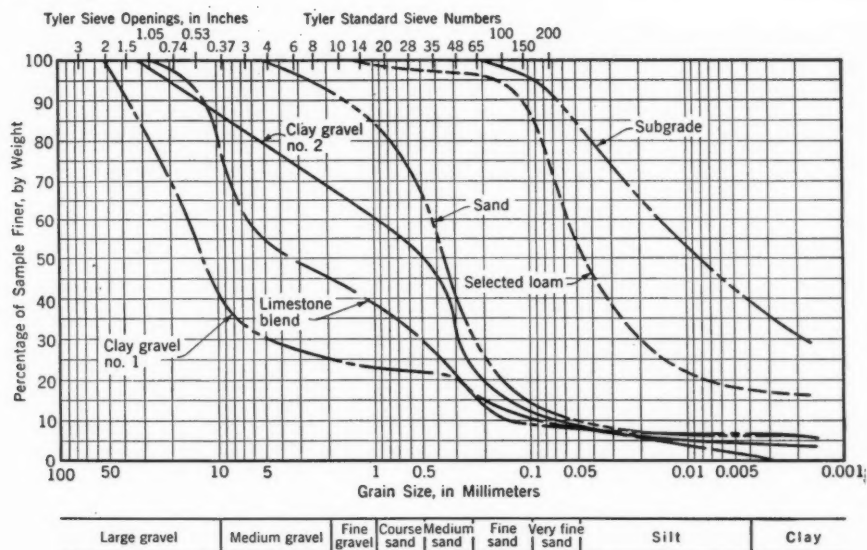


FIG. 16.—MECHANICAL ANALYSIS DIAGRAM FOR CONSTRUCTION MATERIALS (UNITED STATES BUREAU OF SOILS CLASSIFICATION)

<sup>24</sup> Senior Engr., New England Div., Corps of Engrs., Boston, Mass.

<sup>25</sup> "Airfield Pavement Design: Flexible Pavements," *Engineering Manual for War Department Construction*, Office of the Chf. of Engrs., U. S. Army, Washington, D. C., May, 1947, Pt. XII, p. 1, paragraph 2-03.

Information was also desired pertaining to the action of the pavement, base materials, and subgrade under traffic conditions and to other data that might be useful in the development of design methods for flexible pavements. For these reasons, the service behavior tests were conducted in such a manner as

TABLE 2.—PHYSICAL PROPERTIES OF CONSTRUCTION MATERIALS IN THE SUBGRADE BASE AND PAVEMENT

Line	Material	ATTERBERG LIMITS <sup>a</sup>				CLASSIFICATION <sup>b</sup>		COMPACTION <sup>c</sup>		Los Angeles abrasion tests <sup>d</sup>
		Liquid limit, $w_L$	Plasticity index, $I_P$	Shrinkage limit	Linear shrinkage	Casa-grande	PRA	Unit dry weight	Moisture content	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	Limestone blend . . . .	NP	NP	13.0	....	GW	A-3	145	4	46.7
2	Clay-gravel No. 1 . . . .	27.7	15.7	16.8	7.4	GF	A-2	138	6	24.5
3	Clay-gravel No. 2 . . . .	NP	NP	18.9	....	GF-SF	A-3	131	7	26.2
4	Sand . . . . .	NP	NP	21.2	....	SW-SF	A-3	116	10	....
5	Selected loam . . . . .	25.9	7.4	21.5	2.0	ML	A-4	118	13	....
6	Subgrade . . . . .	45.1	27.5	11.7	16.1	CL-CH	A-7-6	114	14	....
7	Soil cement . . . . .	....	....	....	....	....	....	112	14	....
8	Asphaltic concrete <sup>e</sup> . . . .	....	....	....	....	....	....	....	....	....

<sup>a</sup> In Cols. 2 and 3, lines 1, 3, and 4, NP denotes "nonplastic." <sup>b</sup> "Classification and Identification of Soils," by Arthur Casagrande, *Proceedings*, ASCE, June, 1947, p. 783. <sup>c</sup> Modified compaction tests adopted by the American Association of State Highway Officials: In Col. 8, the maximum unit dry weight expressed in pounds per cubic foot; and, in Col. 9, the optimum moisture content expressed as percentages of dry weight. <sup>d</sup> Percentage of wear. <sup>e</sup> For other characteristics, see text.

to afford an opportunity for study of the following in addition to the study of the behavior under traffic: (a) Deflections of the pavement, base, and subgrade under moving and static wheel loads, and under circular bearing plate loads; (b) pressures induced at the surface of the subgrade by moving and static wheel loads; and (c) CBR and plate bearing test procedures and results. Detailed results of the tests were published<sup>26</sup> in 1944.

#### SUBGRADE, BASE MATERIALS, AND PAVEMENT

The physical properties of the test data are summarized in Table 2; grain-size curves of the materials are shown in Fig. 16. A description of the material in Table 2 follows:

Line	Description
1	The limestone blend was a fairly well-graded, soft, crushed limestone with sand and some fines. It was not considered comparable to a hard, well-graded crushed rock.
2	Clay-gravel No. 1 was a fairly well-graded sandy gravel with a plastic binder.
3	Clay-gravel No. 2 was a fairly well-graded gravelly sand with a silty binder.
4	This sand was fairly well graded and silty with a trace of clay.
	Lines 2, 3, and 4 were pit-run materials from local deposits.

<sup>26</sup> "Report on Barksdale Field Service Behavior Tests," U. S. Engr. Office, Little Rock, Ark., October, 1944.

- 5 This material was a sandy clayey silt or a selected loam.
- 6 This material was a medium plastic clay.
- 7 The soil cement was a mixture of a selected raw soil loam the same as item 6, mixed with Portland cement. The tests revealed an average of 8.6% cement by volume. The cement content and the unit dry weight of the soil cement were lower than indicated as necessary by design tests.
- 8 The relative density of the asphaltic concrete was 98.7 and the field stability tests,<sup>27</sup> developed by Prévost Hubbard, Affiliate, ASCE, were performed under a load of 2,148 lb. The extraction test disclosed 6.4% bitumen and the penetration of the residue at 77°F (100 g, 5 sec) was 74.

The aggregate for the asphaltic concrete (line 8, Table 2) was the same as for the limestone blend (line 1), except that all the material was crushed to pass the  $\frac{3}{4}$ -in. screen, the sieve analysis for the mineral aggregate being

Screen	Percentage passing
$\frac{3}{4}$ in. ....	100.0
$\frac{1}{2}$ in. ....	95.7
No. 4. ....	61.8
No. 10. ....	49.2
No. 40. ....	28.3
No. 80. ....	15.3
No. 200. ....	9.1

**Subgrade.**—The subgrade was an alluvial deposit of reddish brown to black, medium plastic clay which was relatively uniform in structure and moisture content. The clay deposit extended to a depth of about 5 ft and was underlain by a deposit of sandy clayey silt which extended to an explored depth of 10 ft. During the traffic tests the water table elevations varied from about 1 ft to 4 ft below the surface of the subgrade. The CBR of the subgrade was between 5% and 7%.

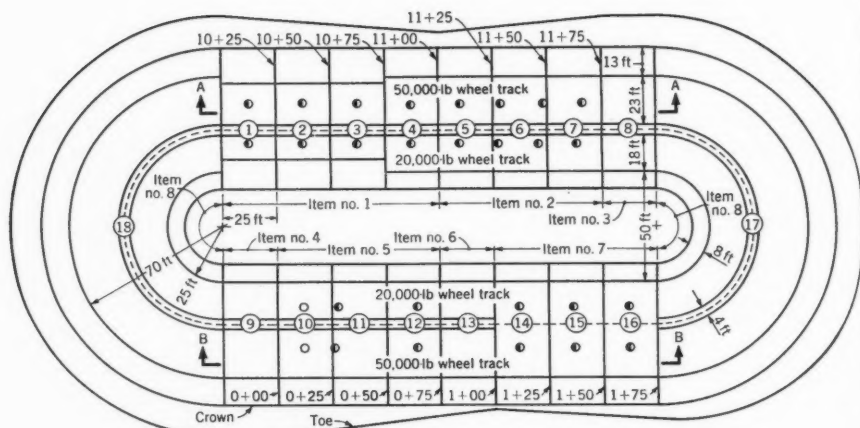
**Base Materials.**—In this paper the term "upper base materials" has been used to indicate the limestone blend, clay-gravels Nos. 1 and 2, and sand (lines 1 to 4, Table 2).

**Pavement.**—The pavement consisted of a 3-in. wearing course of hot-mix asphaltic concrete which was excellent in quality and uniform throughout.

#### TEST PROGRAM

A test section designed in accordance with tentative design curves for 20,000-lb and 50,000-lb wheel loads was constructed on the medium plastic clay subgrade. Extensive tests on the subgrade and on the materials locally available for base course construction were conducted to determine the CBR, the compaction requirements, and the other properties prior to design and construction of the test pavements. The subgrade was stripped level and compacted, and a test section consisting of two test tracks, one for the 20,000-lb

<sup>27</sup> "The Rational Design of Asphalt Paving Mixtures," *Research Series No. 1*, Asphalt Inst., October 15, 1935.



(a) PLAN

LEGEND

Indicates location of:

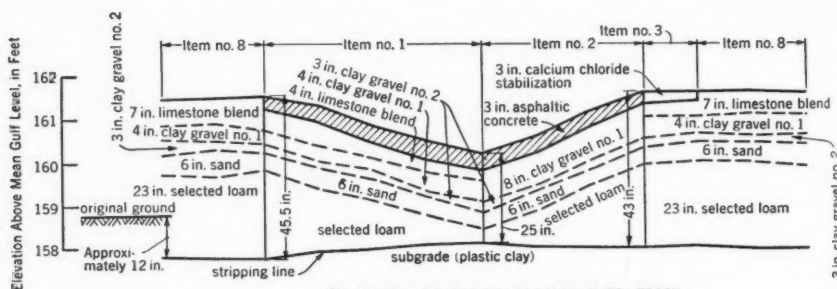
- surface and subgrade deflection plugs
- surface deflection plugs only

Horizontal Scale, in Feet

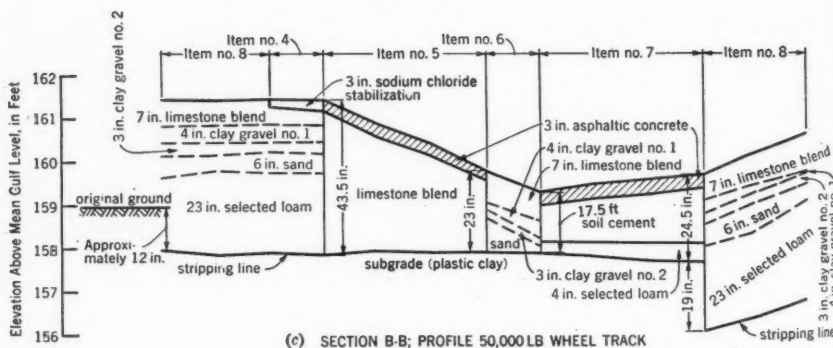


**NOTE:**

NOTE: Layer thicknesses are design values



(b) SECTION A-A; PROFILE 50,000 LB WHEEL TRACK



(c) SECTION B-B: PROFILE 50,000 LB WHEEL TRACK

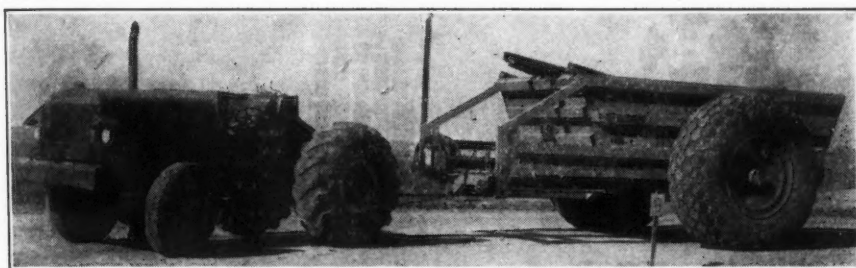
FIG. 17.—PLAN AND PROFILE OF THE TEST SECTION AS CONSTRUCTED

wheel load and the other for the 50,000-lb wheel load, was constructed. The test section was oval in shape with two parallel paved straightaways joined at the ends by semicircular unsurfaced "turnarounds." Inner and outer tracks were constructed for the 20,000-lb and the 50,000-lb wheel loads, respectively. A plan and typical profiles of the test section are shown in Fig. 17. Each test track comprised four different types of bases, as follows: (1) A layered base

TABLE 3.—CHARACTERISTICS OF LOADING EQUIPMENT; SERVICE BEHAVIOR TEST SECTION

Coverage No.	Tracking wheel	Direction of travel	Tire size	Tire air <sup>a</sup>	Contact pressure <sup>a</sup>	Contact area <sup>b</sup>
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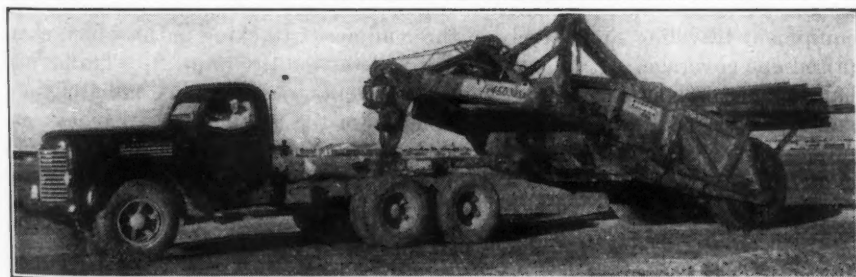
(a) 50,000-LB WHEEL LOAD



EUCLID FOUR-WHEEL RUBBER-TIRED TRACTOR AND LETOURNEAU MODEL R-30 "TOURNATRailer"

0-9	Left	Counterclockwise	30 by 40	60	76.5	653.5
10-1,248	Left	Counterclockwise	30 by 40	50	70.7	707.0
1,249-5,005	Right	Clockwise	30 by 40	50	80.4	621.6

(b) 20,000-LB WHEEL LOAD



GAR WOOD MODEL 515, 14-YD SCRAPER AND INTERNATIONAL MODEL K8-F, SIX-WHEEL TRUCK TRACTOR

0-9	Right	Counterclockwise	18 by 24	60	65.7	304.4
10-321	Right	Counterclockwise	18 by 24	50	63.6	314.2
322-2,621	Left	Clockwise	18 by 24	50	63.5	315.0
2,622-5,009	Left	Clockwise	18 by 24	42	56.6	353.1

<sup>a</sup> Pressure, in pounds per square inch. <sup>b</sup> Square inches.

construction consisting of several layers of different materials with a high CBR material directly under the pavement (item 1, Fig. 17); (2) a layered base construction similar to that in item 1 but with a lower CBR material directly under the pavement (item 2, Fig. 17); (3) a high CBR material for the full thickness of the base (item 5, Fig. 17); and (4) a soil cement base (item 7, Fig. 17). The construction of these items was similar for the 20,000-lb and the 50,000-lb wheel load tracks except that lesser thicknesses of base were used for the smaller wheel load. Variations in thickness of the items were obtained by sloping the surface of the test tracks as shown in the profiles, Figs. 17(b) and 17(c).

Loaded earth-moving equipment (Table 3) with tire contact areas approximately comparable to those of planes with similar wheel loads (Fig. 18) was

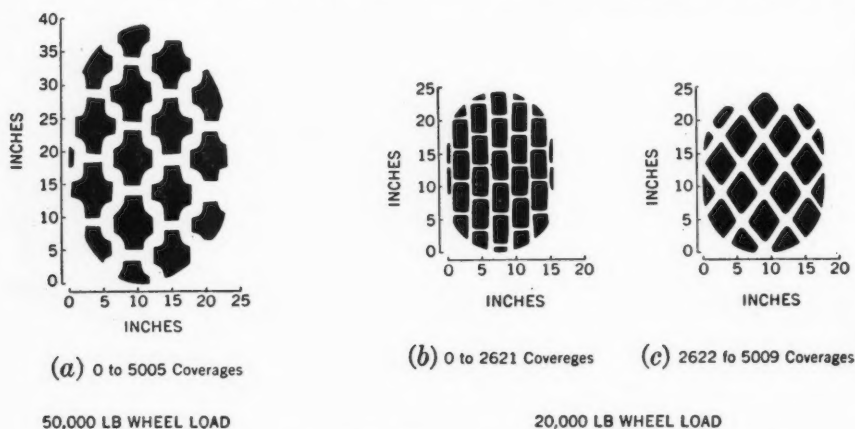


FIG. 18.—TYPICAL TIRE PRINTS OF LOADING WHEELS FOR COVERAGES IN TABLE 3

used to produce the specified wheel loads. The loaded equipment was pulled around each test track by traction units, with one rear wheel of the loaded equipment traveling successively in three adjacent tracking paths which comprised one coverage. Each path was as wide as the tire imprint. Traffic was halted at various intervals for test measurements, observations, maintenance, and repairs to the tracks. After completion of the traffic tests, trenches were excavated in each test track to determine the lateral displacement (shear deformation) and the consolidation of the base and the subgrade. Extensive field and laboratory tests were made on the asphaltic concrete pavement, on all base materials, and on the subgrade to determine behavior under traffic.

#### SUMMARY OF TEST RESULTS

In the analyses of the test results it was assumed that, if no shear deformation could be determined from the test data in the section during 5,000 coverages of a given load, the section would be satisfactory as an airfield pavement and would have adequate thickness of pavement and base to prevent detrimental shear deformation in the subgrade. Also, the base materials would have adequate stability to prevent detrimental shear deformation in the base. It was assumed, furthermore, that, during construction of an actual airfield

pavement, if the soils are compacted to the same relative density as those in the test section after 5,000 coverages, excessive consolidation due to traffic compaction would not occur. Typical cross sections of the lateral displacement, and curvature of the 3-in. asphaltic concrete pavement resulting from repeti-

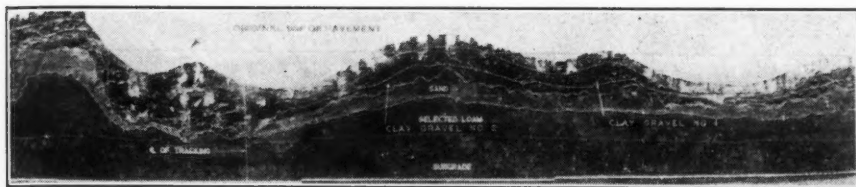


FIG. 19.—DISPLACEMENT AND CURVATURE AT STATION 11 + 45.7, FIG. 17(a)  
(ITEM 2, 20-KIP TRACK; THICKNESS OF PAVEMENT AND BASE, 22.5 IN.)

tions of wheel loads, are shown in Figs. 19 and 20. In this paper, analyses are presented only for the layered bases (items 1 and 2, Fig. 17) and the limestone blend base (item 5) as these items particularly pertain to the development of design criteria for flexible pavements.



FIG. 20.—DISPLACEMENT AND CURVATURE AT STATION 11 + 01, FIG. 17(a)  
(ITEM 2, 50-KIP TRACK; THICKNESS OF PAVEMENT AND BASE, 25.5 IN.)

*Thickness of Pavement and Base Required to Prevent Detrimental Shear Deformation in the Subgrade.*—In Table 4, the 50,000-lb track and the 20,000-lb track, respectively, are compared with regard to the combined thickness of pavement and base required to prevent detrimental shear deformation in the subgrade as determined from the analysis of test data and as obtained by the tentative design curves of the California method.<sup>28</sup> It will be noted that when no shear deformation occurred in the base materials the thicknesses of pavement and base required to prevent detrimental shear deformation in the subgrade in the service behavior test are in substantial agreement with the thicknesses obtained from the tentative design curves (see items 1 and 5, 50,000-lb track, and item 5, 20,000-lb track). Where shear deformation occurred in the base materials, a greater thickness was required to prevent detrimental shear deformation in the subgrade. This fact is shown by comparing the thickness required in item 1, 50,000-lb track, with the thickness required in item 2, 50,000-lb track, and by comparing the thicknesses required in items 1 and 2, 20,000-lb track, with the thickness required in item 5, 20,000-lb track. The thicknesses re-

<sup>28</sup> "Airfield Pavement Design," *Engineering Manual for War Department Construction*, Office of Chf. of Engrs., U. S. Army, Washington, D. C., March, 1943, Pt. II, Chapter XX.

quired for items 1 and 2 for both the 20,000-lb and 50,000-lb tracks are in excess of those required for design when high-stability materials are used in the base. On this basis, the data for all items substantiate the validity of the tentative design curves.

TABLE 4.—SUMMARY OF TEST RESULTS (ITEMS IN FIG. 17)

Line	Description	50,000-Lb TRACK			20,000-Lb TRACK		
		Item 1 <sup>a</sup>	Item 2 <sup>b</sup>	Item 5 <sup>c</sup>	Item 1 <sup>a</sup>	Item 2 <sup>b</sup>	Item 5 <sup>c</sup>
(a) THICKNESS OF PAVEMENT AND BASE REQUIRED TO PREVENT DETRIMENTAL SHEAR DEFORMATION IN THE SUBGRADE							
1	Coverages completed.....	5,000	400	5,000	2,000	2,000	5,000
2	Location of deformation.....	12-ft area <sup>d</sup>	UBM <sup>e</sup>	None <sup>f</sup>	UBM <sup>e</sup>	UBM <sup>e</sup>	None <sup>f</sup>
3	CBR of subgrade.....	5 to 6	6	5 to 6	5 to 6	5 to 7	5
	Pavement and Base Thickness <sup>g</sup> (in Inches) by:						
4	Service behavior tests.....	30 to 36	32 to 37	29 to 34	20 to 25	21 to 26	18 to 21
5	Tentative design curves....	27.5 to 31.0	27.5	27.5 to 31.0	19 to 21	17 to 21	21
(b) REQUIREMENTS FOR LAYERED BASE DESIGN							
6	Coverages completed.....	5,000	400	....	2,000	2,000	....
7	Adequacy of selection <sup>h</sup> .....	Yes	Yes	....	Yes	Yes	....
8	CBR of selected loam.....	13	13	....	10	10	....
	Pavement and Base Thickness (in Inches) by:						
9	Service behavior tests.....	20	20	....	14	14	....
10	Tentative design curves....	17	17	....	14	14	....
(c) QUALITY OF BASE MATERIALS REQUIRED DIRECTLY UNDER THE PAVEMENT							
11	Thickness of layer <sup>i</sup> .....	4(7) <sup>j</sup>	8	24 to 43	2	5	11 to 29
	CBR:						
12	Before tracking.....	42(42) <sup>j</sup>	21	86 to 90	24	28	58 to 64
13	After tracking.....	86(86) <sup>j</sup>	64	90 to 118	42	12	62 to 68
14	Adequacy.....	No(Yes) <sup>j</sup>	No	Yes	No	No	Yes

<sup>a</sup> Item 1 (see Fig. 17); layered base with a limestone blend (line 1, Table 2) directly beneath the pavement.

<sup>b</sup> Item 2 (see Fig. 17); layered base with clay-gravel No. 1 (line 2, Table 2) directly beneath the pavement.

<sup>c</sup> Item 5 (see Fig. 17); limestone blend (line 1, Table 2) for the entire depth of the base. <sup>d</sup> The upper base materials were just adequate in the 12-ft area between the points where the combined thickness of pavement and base was 41 in. and 43 in. They were inadequate in the remainder of item 1. <sup>e</sup> UBM is an abbreviation of "upper base materials." <sup>f</sup> No detrimental shear deformation in the base. <sup>g</sup> Total thickness of pavement and base required to prevent detrimental shear deformation in the subgrade.

<sup>h</sup> Adequacy of the base above selected loam with respect to the thickness required to prevent detrimental shear deformation. <sup>i</sup> Thickness of layer, in inches. <sup>j</sup> Values in parentheses refer to the 12-ft area discussed in footnote <sup>d</sup>.

*Effect of Quality of Base Materials on the Total Required Thickness of Pavement and Base.*—Referring again to Table 4(a), it will be noted that the required thickness of pavement and base for layered base construction (item 1, 50,000-lb track) is slightly greater than the thickness for the base with limestone blend for the full depth (item 5, 50,000-lb track). Although the thickness required in the service behavior test was slightly greater in item 1 than in item 5, it is believed that the difference can be considered as less than 5% when the effect of shear deformation in the base of item 1 on the total thickness is considered. Although the pavement surfaces were in better condition after traffic on the parts of the test section where the base was composed of limestone

blend for the full depth, from the point of view of prevention of shear deformation in the subgrade the layered base performed essentially as well as did the full depth of high-quality base course material.

*Requirements for Layered Base Design.*—Table 4(b) shows a comparison of the thickness of pavement and base required in layered base construction to prevent shear deformation in a definite layer with the thickness obtained from the tentative design curves. The thickness required in these tests is based on an analysis of the behavior of the selected loam.

It will be noted in Table 4(b) and Fig. 17 that the thicknesses of pavement and base above the selected loam in items 1 and 2 were the same from the thin end of the items to the thick end. The results of the service behavior tests indicate that the thicknesses used were adequate; however, it is possible that lesser thicknesses might have been adequate to prevent detrimental shear deformation in the selected loam. Although a definite conclusion cannot be made regarding the validity of the design curves to determine the qualities and the thicknesses of each layer of base, the data do indicate that the tentative design curves are substantially valid.

TABLE 5.—COMPACTION REQUIREMENTS TO PREVENT EXCESSIVE CONSOLIDATION

Line	Description	ITEM 1				ITEM 2				ITEM 5			
		Upper base materials	Se-lected loam	Subgrade at:		Upper base materials	Se-lected loam	Subgrade at:		Limestone blend at:		Subgrade at:	
				Sur-face	1-ft depth			Sur-face	1-ft depth	Sur-face	1-ft depth	Sur-face	1-ft depth
(a) 50,000-LB TRACK													
	Modified AASHO <sup>a</sup> Maximum Unit Dry Weight (%):												
1	Before tracking.....	97 to 101	89	88	86	93 to 100	92	87	88	96	94	89	91
2	After tracking.....	97 to 105	93	91	88	101 to 103	95	89	88	99	94	93	90
3	Depth of sampling, in inches <sup>b</sup> ...	....	20	41	53	....	20	42	54	..	15	41	53
(b) 20,000-LB TRACK													
	Modified AASHO <sup>a</sup> Maximum Unit Dry Weight (%):												
4	Before tracking.....	83 to 99	91	91	88	93 to 101	91	84	88	97	88	89	87
5	After tracking.....	92 to 100	93	89	90	95 to 102	93	86	86	97	91	89	88
6	Depth of sampling, in inches <sup>b</sup> ...	....	14	28	40	....	14	27	39	..	15	26	38
<sup>a</sup> American Association of State Highway Officials. <sup>b</sup> Location below the surface of the pavement.													

<sup>a</sup> American Association of State Highway Officials. <sup>b</sup> Location below the surface of the pavement.

*The Quality of Base Materials Required Directly Under the Pavement.*—The data on the adequacy of the base materials directly below the 3-in. high-quality asphaltic concrete pavement from the standpoint of shear deformation for both the 50,000-lb and the 20,000-lb tracks are summarized in Table 4(c). At least 6 in. of material with a CBR of 80% or greater is required directly below a 3-in. asphaltic concrete pavement for a 50,000-lb wheel load. For the 20,000-lb

wheel load, the data are not sufficient to determine the minimum thickness of layer required for material directly below a 3-in. asphaltic concrete pavement; however, the test results indicate that a material with a CBR of 60% for the full depth of base below the pavement is adequate, and may be more than adequate. If the pavement were thinner than 3 in., or not of good quality, it is possible that a material with a CBR greater than 60% would be required for a 20,000-lb wheel load.

*Compaction Requirements to Prevent Excessive Consolidation.*—The compaction attained during construction and the additional compaction resulting from traffic are shown in Table 5. A study of the data indicates that compaction to about 103% modified AASHO maximum unit dry weight was required in the upper base materials for the 50,000-lb track, whereas 100% probably would have been adequate for the 20,000-lb track. The data also indicate that compaction to 95% would have been adequate for the subgrade and the selected loam for either the 20,000-lb or the 50,000-lb wheel loads. The measurable effect of compaction from traffic extended to depths of from 27 in. to 40 in. from the surface of the pavement at the thick ends of the items in the 20,000-lb track and from 41 in. to 53 in. from the surface of the pavement at the thick ends of the items in the 50,000-lb track. The depth of traffic compaction was about 6 in. in the subgrade for the 20,000-lb track and about 12 in. in the subgrade for the 50,000-lb track at the thickness of pavement and base required to prevent detrimental shear deformation in the subgrade (29 in. to 37 in. for 50,000-lb track and 18 in. to 26 in. for 20,000-lb track).

#### CONCLUSIONS

*General.*—On the basis of the results obtained from the service behavior tests and of the assumptions made regarding the significance of the data with respect to the requirements for the quality of pavement desired, the following general conclusions are presented. These conclusions are applicable to the design of flexible type airfield pavements when the subgrade is a medium plastic clay.

1. In general, the service behavior tests at Barksdale Field have substantiated the CBR tentative design curves<sup>28</sup> when applied to flexible pavements constructed on medium plastic clay subgrades.

2. The stresses induced by a 50,000-lb wheel load require that 6 in. of material, with a CBR of 80% or more, be placed directly below a 3-in. high-quality asphaltic concrete pavement. The minimum thickness of layer required for material directly below the pavement was not determined for a 20,000-lb wheel load; however, the data indicate that for this wheel load a material with a CBR of 60% for the full depth of base below a 3-in. asphaltic concrete pavement was adequate and may be more than adequate. If the pavement were thinner, or not of good quality, it is possible that the CBR of the underlying base material should be greater than 60%.

3. The compaction requirements indicated as necessary to prevent excessive consolidation in the base materials and the subgrade under the traffic of 50,000-lb and 20,000-lb wheel loads are as follows:

(a) Base Materials.—For the 50,000-lb wheel load, the upper base materials (limestone blend, clay-gravel No. 1, clay-gravel No. 2, and sand for layered base construction or the 18 in. directly below the pavement for limestone blend for full thickness of base) should be compacted to a minimum of 103% of the modified AASHO maximum unit dry weight. For the 20,000-lb wheel load, the upper base materials (limestone blend, clay-gravel No. 1, clay-gravel No. 2, and sand for layered base construction or the 12 in. directly below the pavement for the limestone blend for full thickness of base) should be compacted to a minimum of 100% of the modified AASHO maximum unit dry weight. The lower base materials (below 12-in. depth) should be compacted to a minimum of 95% of the modified AASHO maximum unit dry weight.

(b) Subgrade.—For both the 20,000-lb and the 50,000-lb wheel loads, the plastic clay subgrade (if the stability can be increased by compaction) should be compacted to a minimum of 95% of the modified AASHO maximum unit dry weight. The depth of compaction should be 12 in. for a 50,000-lb wheel load and 6 in. for a 20,000-lb wheel load.

## WHEEL LOAD TESTS, MARIETTA, GA.

BY JOHN M. GRIFFITH,<sup>29</sup> ASSOC. M. ASCE

When fully loaded, the B-29 Superfortress has a gross weight of approximately 120,000 lb, which is carried on dual wheel landing gear. When this plane was first introduced, the CBR method of design did not contain flexible pavement design criteria for dual wheel loads. Therefore, the Chief of Engineers, United States Army, was requested by Headquarters, Army Air Forces, to study the problem and to formulate design criteria.

The Office of the Chief of Engineers, through the Engineer Board, directed the Waterways Experiment Station to design, construct, and conduct tests at Marietta on a test section composed of flexible pavements on various types of bases. The primary purpose of the test section was to furnish data whereby design curves, similar to the tentative CBR design curves<sup>1</sup> could be developed for a dual wheel load of 60,000 lb. The application of the data to the development of design curves for the B-29 dual wheels is given in Symposium paper No. 10. The test section was also designed to furnish other subordinate information; but only a limited amount of data other than those required to accomplish the primary purpose is discussed in this paper because of necessary space limitations. The complete results of all tests conducted at Marietta are presented in a report of the Waterways Experiment Station.<sup>30</sup>



FIG. 21.—BOEING B-29 SUPERFORTRESS (XB-118,335) USED FOR PAVEMENT BEHAVIOR TESTS

The test program at Marietta was conducted to determine the relationship between subgrade deflections and pressures under dual and single wheel loads so that thickness design curves similar to those used in the CBR method for single wheels could be prepared. In these tests, B-29 and B-24 planes were used to impose the desired dual and single wheel loads. Fig. 21 shows the B-29 plane used in the tests. Accelerated traffic testing was not contemplated

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<sup>30</sup> "Certain Requirements for Flexible Pavement Design for B-29 Planes," U. S. Waterways Experiment Station, Vicksburg, Miss., Appendix on "Flexible Pavement Tests, Marietta, Ga.," 1 August 1945.

in the event that these data proved adequate. The test section was designed, however, so that accelerated traffic tests could be conducted should they be considered desirable.

#### DESCRIPTION OF TEST SECTION

The test section was constructed as shown in Figs. 22 and 23 with ordinary construction equipment using standard construction procedures; however, close field control and inspection were maintained throughout construction to insure uniform conditions and close adherence to the desired design requirements. The in-place soil was a highly elastic micaceous silty clay that was considered unsuitable for the purpose of the test; therefore, an imported cohesive clay was placed as a subgrade beneath all the test section except the sand-asphalt item. The average CBR of the imported clay subgrade as constructed was about 4%. A clean well-drained sand was placed as the subgrade for the sand-asphalt item. The macadam bases were constructed of a hard, durable, crushed rock, in 4-in. layers, choked with limestone screenings, which produced a base with a CBR of 80% or more. The telford base was constructed of limestone rock averaging about 10 in. in height and not less than 3 in. in width. The stones were bedded into a sand cushion course over the clay subgrade. Voids between the stones were filled with crushed limestone spalls and screenings. The sand-clay base was constructed of a fairly well-graded sand-clay material having a plasticity index of about 5%. The average CBR of the sand-clay base as constructed was about 35%.

The telford and macadam items were surfaced with a 2-in. thickness of hot-mix asphaltic concrete of good quality. The sand-clay item was surfaced with a 1-in. thickness of the same asphaltic concrete and on this a steel pierced plank landing mat was laid. The sand-asphalt item was surfaced with 6 in. of hot-mix sand asphalt laid directly on the sand subgrade.

Deflection gages were located in the test section as shown in Fig. 22. To obtain the desired information, it was necessary to install two gages at each location; one gage furnished a measurement of the total vertical movement or deflection of the surface of the pavement, and the other deflection gage measured the compression in the base and wearing course. By subtracting the latter from the former deflection, the amount of deflection of the surface of the subgrade was determined. Only a single deflection gage to measure total surface deflection was installed in the telford item since it was considered that the single layer of telford stone would be incompressible. Pressure cells were installed at four elevations, both in the clay subgrade under the medium thickness of the macadam base item and in the sand subgrade of the sand-asphalt item.

Immediately on completion of construction and prior to any plane test runs, 20 coverages of a 20,000-lb wheel load were applied to all items except the telford. The objective of these initial coverages was to consolidate the test items to reduce the amount of consolidation appearing in the measured deflections. An attempt was made to consolidate the telford item with a 10,000-lb wheel load, but traffic was discontinued after the first coverage because of visibly large deflections.

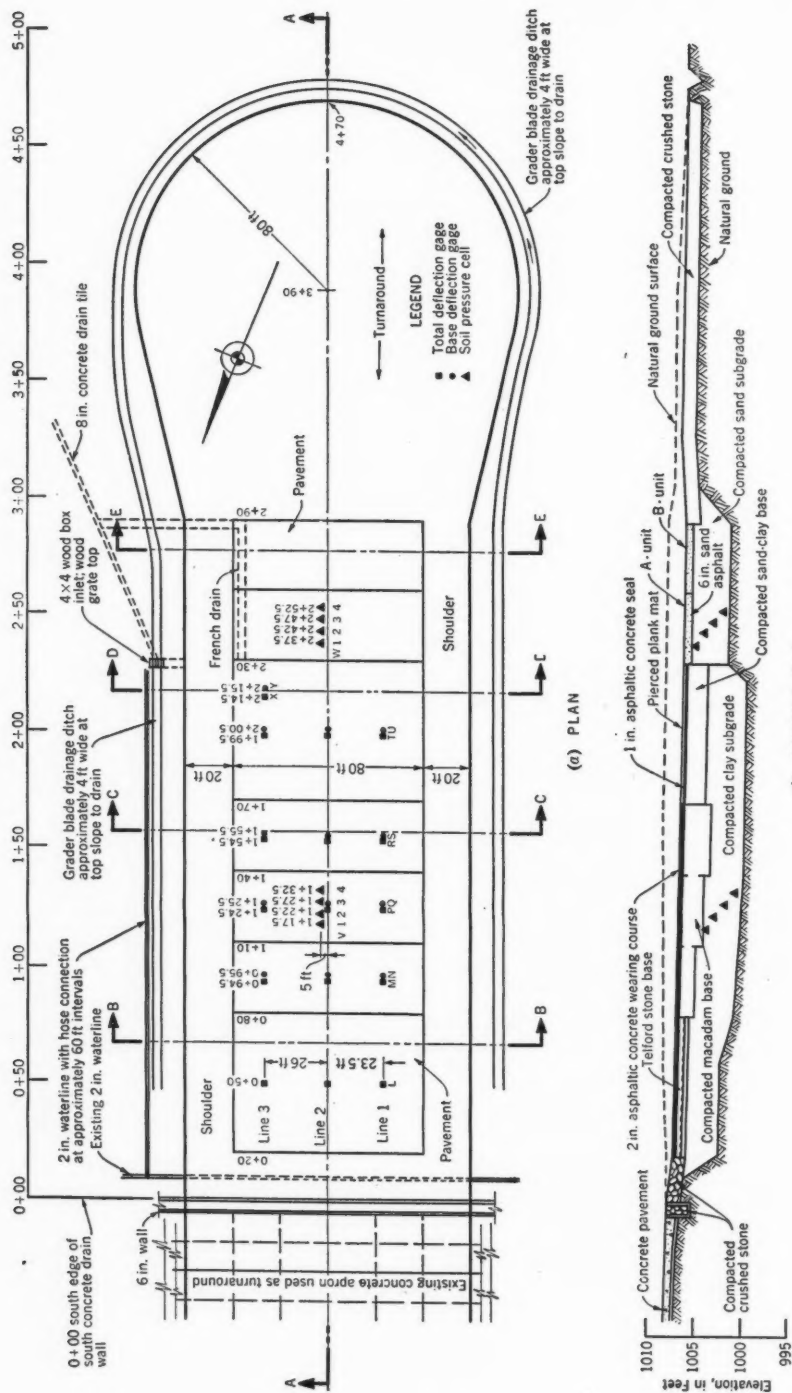


FIG. 22.—FINAL PLAN AND PROFILE, TEST SECTION, MARIETTA, GA.

By reference to Fig. 22(a) it may be noted that there are three longitudinal lines of deflection gages, designated lines 1, 2, and 3. Transverse lines of gages are identified by letters. The three longitudinal lines of gages were installed to furnish one separate line of gages each for the single and dual wheel loads (lines 3 and 1, respectively) and a common line of gages for both loads (line 2).

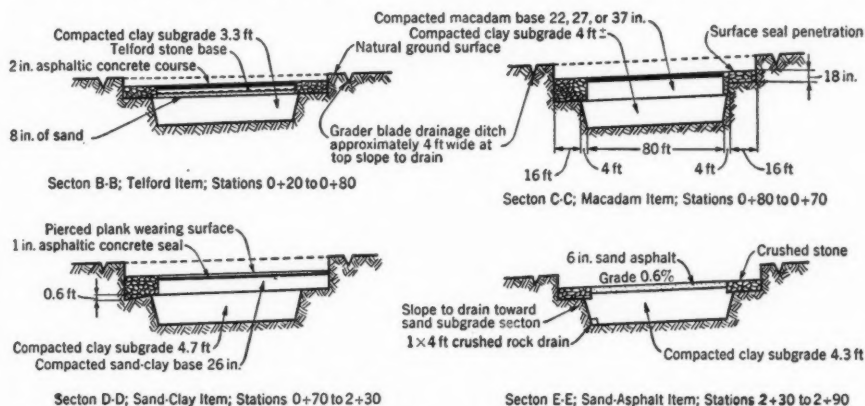


FIG. 23.—TYPICAL CROSS SECTIONS, TEST SECTION, MARIETTA, GA. (PLAN AND PROFILE IN FIG. 22)

On completion of the single wheel load tests, however, dual wheel tests were conducted over line 3 in addition to those on lines 1 and 2 for purposes of further correlation.

#### TESTS CONDUCTED

Deflection and pressure measurements were made under three loads for both the single wheel and the dual wheel. These loads were 20,000 lb, 25,000 lb, and 30,000 lb for the single wheel and 40,000 lb, 50,000 lb, and 60,000 lb for the dual wheel. It may be noted that the dual wheel loads were twice those on the single wheel; thus, the load on each tire of a dual assembly was the same as the single wheel loads. Deflections and pressures induced by both the single and dual wheels were measured. Initially, measurements were made for the following conditions: (a) Plane standing with motors dead, (b) plane standing with motors running about 1,000 rpm, (c) plane towed with motors dead, and (d) plane taxiing. After a part of the program was completed, it was determined that the deflections and pressures under some of the conditions were practically equal; therefore, certain conditions were deleted in the latter phases of the tests. Deflection data obtained for the single wheel loads of a B-24 plane for the foregoing conditions and similar data for the dual wheel loads of a B-29 plane, with one tire of the duals over the gage and with the center of the dual wheels over the gage, are shown in Table 6. Pressure data for these conditions are presented in Table 7 for the single (B-24) and dual (B-29) wheel load conditions. Each deflection or pressure measurement shown in Table 6 and 7 is the average of three or more individual tests.

#### INSTRUMENTATION

Deflection gages and pressure cells used in these tests were designed and constructed by personnel of the Waterways Experiment Station. General

operational features of the deflection gage and the method of their installation in the pavement are illustrated in Fig. 24, a deflection gage sleeve being mounted in the pavement and base as shown. Where total surface deflection measurements were desired, a  $\frac{3}{4}$ -in. pipe reference was driven deeply into the subgrade and capped by a 2-in.-diameter steel gage foot. The pipe reference for base compression measurements was grouted into the base in the manner shown. The deflection gages were bolted into the gage sleeves which in turn were firmly fixed in the pavement by slotted flanges. Pavement deflections caused downward movements of the gages with respect to the gage foot, thus compressing the deflection gage. The mechanics of the deflection gage are illustrated in plan and cross section in Fig. 25 where it may be noted that compres-

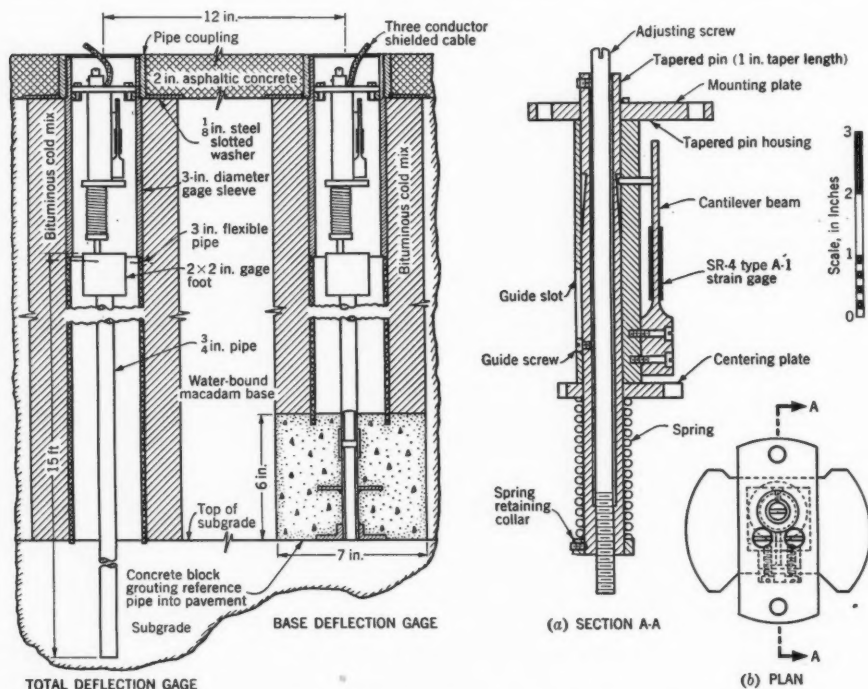


FIG. 24.—TYPICAL DEFLECTION GAGE INSTALLATION      FIG. 25.—DETAILS OF DEFLECTION GAGE

sion of the gage causes movement of a tapered pin within the external shell of the gage. The pin shown (1-in. taper length) is interchangeable with a 0.3-in. taper length, the latter providing greater accuracy where deflections less than 0.3 in. were anticipated. Movement of the tapered pin causes bending in a cantilever beam which has two resistance strain gages mounted as shown. Bending of the cantilever thus causes a change in the resistance of the strain gages, cumulative for the two gages, which is measured in a Wheatstone bridge circuit. Prior to field use each gage was calibrated in the laboratory in a device in which the tapered pin was moved by measured increments and the change in resistance of the strain gages was determined.

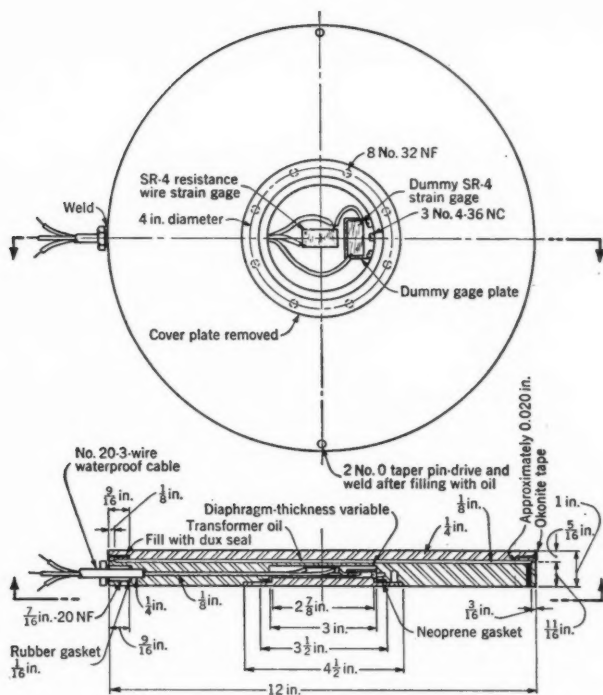
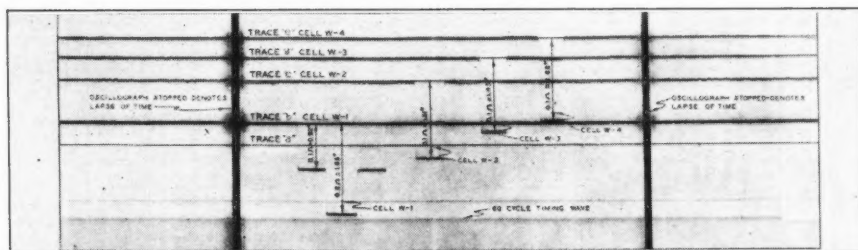
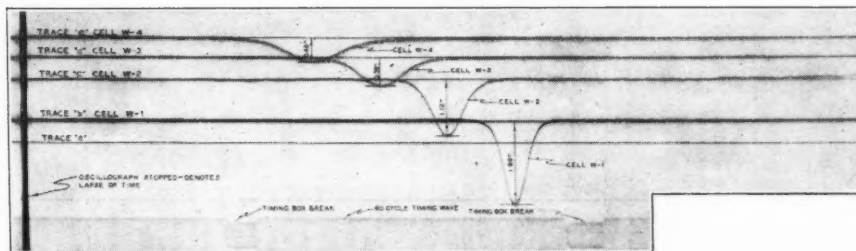


FIG. 26.—EARTH PRESSURE CELL



(a) CALIBRATION PRIOR TO TEST



(b) MOVING PLANE TEST OVER PRESSURE CELLS

FIG. 27.—TYPICAL MOVING WHEEL LOAD OSCILLOGRAM



[illegible]

<sup>a</sup> Line Nos. refer to Fig. 22. <sup>b</sup> Approximately 1,000 rpm. <sup>c</sup> No reading; gage inoperable. <sup>d</sup> Gages in line 2, Table 6(e), were destroyed by previous tests with the B-29 plane. These were replaced as additional gages in line 3.

TABLE 7.—SUMMARY OF PRESSURE CELL DATA IN THE FLEXIBLE  
(Average Recorded Pressures,

No.	Test conditions of plane	SAND-ASPHALT ITEM <sup>a</sup>												42 in. N	
		W-1			W-2			W-3			W-4				
		30 in. N	Over	30 in. S	30 in. N	Over	30 in. S	30 in. N	Over	30 in. S	30 in. N	Over	30 in. S		
	Depth of Pressure Cell Below Surface—														
1	Before testing.....	10.3 in.			24.5 in.			37.9 in.			51.2 in.				
2	After testing.....	10.7 in.			23.8 in.			37.7 in.			51.0 in.				
(a) SINGLE WHEEL															LOADS
	Plane Empty; 20-Kip Load on Each Single Wheel:														
3	Towed, motors dead.....	42.4	...	...	18.4	...	...	8.6	...	...	5.6	...	...		
4	Taxiing.....	30.4	...	...	14.5	...	...	6.2	...	...	4.4	...	...		
	Standing—														
5	Motors running 1,000 rpm.....	36.9	...	...	15.8	...	...	6.8	...	...	4.5	...	...		
6	Motors dead.....	1.0	42.7	0.9	3.2	20.7	3.0	2.8	9.1	2.1	2.6	5.7	2.3	1.6	
	Plane Partly Loaded; 25-Kip Load on Each Single Wheel:														
7	Towed, motors dead.....	48.5	...	...	23.5	...	...	10.3	...	...	5.6	...	...		
	Standing—														
8	Motors running 1,000 rpm.....	44.2	...	...	22.2	...	...	9.3	...	...	...	...	...		
9	Motors dead.....	47.9	2.4	2.4	23.3	2.0	2.4	10.8	2.6	2.8	6.7	3.0	...	1.6	
	Plane Loaded; 30-Kip Load on Each Single Wheel:														
10	Towed, motors dead.....	52.3	...	...	26.9	...	...	12.0	...	...	7.9	...	...		
11	Taxiing.....	49.2	...	...	25.7	...	...	10.8	...	...	6.9	...	...		
	Standing—														
12	Motors running 1,000 rpm.....	51.5	...	...	28.3	...	...	11.5	...	...	7.9	...	...		
13	Motors running 2,500 rpm.....	...	...	...	22.0	...	...	9.6	...	...	4.8	...	...		
14	Motors dead.....	1.6	52.7	2.4	4.1	26.2	4.2	3.1	12.9	2.8	4.2	8.2	2.4	1.8	
(b) DUAL WHEEL															LOADS
	Plane Empty; 40-Kip Load per Dual Wheel:														
	Towed, with Motors Dead—														
15	One tire over cell.....	27.4	...	...	16.4	...	...	6.7	...	...	5.7	...	...		
16	Center of dual wheel over cell.....	8.8	...	...	8.5	...	...	7.4	...	...	6.5	...	...		
17	Taxiing, with one tire over cell.....	29.3	...	...	17.6	...	...	7.5	...	...	5.4	...	...		
	Standing, Motors Dead—														
18	One tire over cell.....	1.2	42.4	1.0	1.4	18.1	1.8	2.7	9.5	2.5	3.7	6.5	3.0	2.4	
19	Center of dual wheel over cell.....	0.0	3.6	0.2	1.1	9.8	2.0	2.9	8.6	3.2	3.7	7.4	3.2	2.7	
	Plane Loaded; 60-Kip Load per Dual Wheel:														
	Towed, with Motors Dead—														
20	Tire over cell.....	55.9	...	...	30.3	...	...	13.6	...	...	10.0	...	...		
21	Center of dual wheel over cell.....	16.0	...	...	15.5	...	...	10.4	...	...	9.3	...	...		
22	Taxiing, one tire over cell.....	...	...	...	...	...	...	...	...	...	...	...	...		
	Standing, One Tire Over Cell—														
23	Motors running 1,000 rpm.....	56.8	...	...	26.8	...	...	14.6	...	...	10.2	...	...	2.0	
	Standing; Motors Dead—														
24	One tire over cell.....	1.5	58.2	1.8	2.4	29.9	4.4	3.9	14.3	4.6	5.2	10.5	3.9	2.3	
25	Center of dual wheel over cell.....	0.6	8.0	1.1	2.6	15.8	2.6	4.0	11.6	4.1	5.5	10.7	4.7	2.9	

<sup>a</sup> Pressure cells are identified by letter prefixes W and V (see Fig. 22); the letters N and S refer to the distance actually 1 ft north of each V-cell. This did not affect moving tests as the maximum pressure shown on the oscillogram V-cell and consequently did not show maximum pressure. <sup>b</sup> The numerals in parentheses under "Medium" by N. M. Newmark, M. ASCE, for the wheel load directly over each cell and 1 ft north of each cell and by V-cells need no correction because these distances were corrected to show the true relationship to the cells.

PAV.  
in Po

LOADS

LOADS

north o  
grams v  
Macade  
adding

PAVEMENT BEHAVIOR TESTS, MARIETTA, GA.  
in Pounds per Square Inch)

MEDIUM MACADAM ITEM <sup>a</sup>												No.
V-1			V-2			V-3			V-4			
42 in. N	Over <sup>b</sup>	18 in. S	42 in. N	Over <sup>b</sup>	18 in. S	42 in. N	Over <sup>b</sup>	18 in. S	42 in. N	Over <sup>b</sup>	18 in. S	
34.4 in. 33.0 in.			47.6 in. 46.3 in.			61.2 in. 59.1 in.			74.2 in. 72.0 in.			1 2

## LOADS (B-24 PLANE)

...	...	8.7	...	...	5.7	...	...	2.8	...	...	1.8	...	3
...	...	8.5	...	...	6.0	...	...	2.7	...	...	1.8	...	4
...	...	6.2(8.1)	...	...	4.0(4.7)	...	...	2.2(2.5)	...	...	1.6(1.7)	...	5
2.3	1.6	7.5(9.4)	4.9	0.9	4.3(5.0)	3.1	0.6	2.2(2.5)	1.6	1.3	2.1(2.2)	1.7	6
...	...	11.2	...	...	6.8	...	...	3.7	...	...	2.7	...	7
...	...	7.8(10.2)	...	...	5.2(6.0)	...	...	2.4(2.8)	...	...	2.4(2.5)	...	8
3.0	1.6	8.1(10.5)	6.5	1.2	5.2(6.0)	5.0	0.9	2.7(3.1)	2.3	1.2	2.2(2.3)	1.8	9
...	...	12.3	...	...	8.2	...	...	4.5	...	...	3.2	...	10
...	...	11.4	...	...	7.6	...	...	4.2	...	...	2.8	...	11
...	...	10.4(13.2)	...	...	6.2(7.2)	...	...	3.9(4.3)	...	...	2.8(2.8)	...	12
...	...	10.8(13.6)	...	...	6.4(7.4)	...	...	3.7(4.1)	...	...	2.4(2.5)	...	13
2.4	1.8	10.7(13.5)	6.9	2.2	7.0(8.0)	5.8	1.4	4.0(4.4)	3.3	1.6	3.1(3.2)	2.5	14

## LOADS (B-29 PLANE)

...	...	10.3	...	...	6.9	...	...	3.6	...	...	3.0	...	15
...	...	10.6	...	...	7.8	...	...	4.1	...	...	3.6	...	16
...	...	9.7	...	...	6.3	...	...	3.2	...	...	2.2	...	17
5.3	2.4	8.5(10.6)	7.7	1.9	5.5(6.3)	4.3	1.9	3.8(4.1)	3.6	2.3	3.5(3.6)	3.3	18
4.3	2.7	8.5(10.1)	6.9	2.3	5.5(6.4)	4.7	1.9	3.4(3.9)	3.4	1.7	3.3(3.5)	3.5	19
...	...	43.0	...	...	9.0	...	...	4.9	...	...	5.0	...	20
...	...	15.4	...	...	11.3	...	...	6.8	...	...	6.3	...	21
...	...	14.1	...	...	10.2	...	...	6.7	...	...	5.8	...	22
2.2	2.0	11.4(14.5)	10.9	2.9	9.4(10.6)	6.3	3.4	6.4(7.0)	5.3	2.7	5.2(5.4)	4.2	23
5.3	3.9	11.7(14.8)	10.0	3.1	8.8(10.0)	6.9	2.9	4.9(5.5)	5.0	2.2	4.9(5.1)	4.8	24
7.7	4.7	10.2(12.6)	10.0	3.7	9.6(10.9)	5.5	4.5	6.0(6.7)	5.8	3.4	6.1(6.5)	4.5	25

distance north or south of the cell indicated. Surface points supposedly directly above V-cells were, by erroneous location, grams was recorded; but on standing tests the plane was stopped over the surface points 1 ft away from each Medium Macadam Item" are the corrected values obtained by computing pressures by the graphical solution introduced adding the difference to the observed readings. Readings shown for distances 42 in. north and 18 in. south of cells.

General features of the pressure cells used in these tests are shown in Fig. 26. The cell contains a machined steel diaphragm to which is attached a resistance type strain gage. Pressure is transmitted from the face plate to the diaphragm by an oil-filled chamber. Pressure applied to the diaphragm causes a strain resulting in a resistance change in the strain gage. The effect of variation in strain-gage temperature is compensated for by use of a dummy strain gage which is the same as that mounted on the diaphragm. The dummy gage is mounted in the chamber and is not affected by diaphragm strains but only by temperature changes. It is placed in the electrical circuit in such a manner as to nullify the effects of temperature changes in the active strain gage. Pressure cells are calibrated in the laboratory prior to field use by being placed in a calibrating device in which known increments of pressure may be applied uniformly over the face of the cell through an air-filled rubber diaphragm. In this manner, changes in applied pressure may be related to changes in resistance of the strain gage.

Field measurements of deflections and pressures were made by recording oscillographs and associated electrical circuits which furnished permanent records of the resistance changes with application of loads. A typical oscillograph record (oscillogram) is shown in Fig. 27 for four of the pressure cells; deflection-gage records are similar. Known resistance changes indicated by decade resistors in the circuit furnish a method of converting scaled changes in the position of the trace on the oscillogram to changes in deflection or pressure. A 60-cycle timing wave is recorded on the bottom of the record, and changes in the amplitude of this wave indicated the passage of the wheel load over a timing box on the test section, the location of which was known. Thus, the speed of the moving wheel load could be determined.

#### DISCUSSION OF TESTS AND TEST RESULTS

Since accelerated traffic testing was not conducted, the pavement items were subjected to relatively light traffic when compared to that imposed on other test sections where accelerated traffic was applied. Test traffic was applied in an expedient manner for obtaining the desired deflection and pressure data, and the traffic pattern was not uniform as in accelerated traffic tests. The amount of applied traffic (in addition to the consolidation traffic discussed in a preceding paragraph) therefore can only be estimated, and is approximately as follows: (a) Line 1 received 50 coverages with the dual wheel over a traffic lane 6 ft in width, this being the width required to meet the test conditions for one wheel over the gage and for the center of dual wheels over the gage; (b) line 2 received 150 coverages, about 40% of which was first applied by the dual wheel over a lane 6 ft in width and the remainder was concentrated in a single track directly over the gage line for the single (B-24) wheel load traffic; and (c) line 3 received 120 coverages, about 70% of which was first applied in a single track by the single wheel load traffic, and the remainder was applied over about a 6-ft lane with the dual wheel traffic as previously described.

The three macadam items all successfully withstood the limited amount of traffic imposed during these tests. By reference to Table 6, it may be noted that, although there are variations in deflections as measured on the three

gage lines of a given item, there are apparent overall-trends of logical deflection variation with load and with base thickness. It may also be noted that in general the condition of standing with motors dead is about as severe as any condition encountered. Deflections in the thin macadam item are considered to have been sufficiently high that the item probably would not have stood an indefinite number of traffic coverages. Pressure data shown in Table 7 and obtained in the subgrade under the medium macadam base compare favorably with check computations made in accordance with the theory of elasticity, and these data are believed to be valid. The CBR of the surface of the clay subgrade beneath the macadam items averaged about 4% as constructed, and at the completion of all testing the CBR of this clay subgrade still averaged about 4% at the surface. The moisture content of the surface of the clay subgrade averaged about 30% before and after plane testing. The dry density of this subgrade increased from an average of 89 lb per cu ft before plane testing to 92 lb per cu ft after plane testing. The average density of the macadam bases was about 140 lb per cu ft after all testing, and the CBR of these bases averaged about 80% throughout the test period. It may be noted that the tentative CBR design curves, presented elsewhere in this Symposium, require a total thickness of base and pavement of about 29 in. for a single wheel load of 30,000 lb. It is believed probable, therefore, that the thin macadam item would not have been satisfactory in an accelerated traffic test. Under the limited amount of traffic applied in these tests, however, only minor rutting occurred in this item and the ruts were limited primarily to the junction of the thin macadam item with the telford item.

The telford item was unsatisfactory for wheel loads of the magnitude of those used in these tests, and it was previously noted that visibly large deflections occurred under only a 10,000-lb wheel load. B-29 tests were first applied on line 1 of this item, and surface deflections of about 1 in. were measured on the first few trials after which the deflections increased and exceeded the 1-in. measuring capacity of the deflection gage. Deterioration of the telford item was rapid, and deep ruts and apparent lack of stability caused early abandonment of the scheduled testing. After a few test runs, oak planking was laid over the wheel lanes to provide access to the other items. The telford pavement reacted similarly under the single wheel loads of the B-24. One feature that should be noted is that free water became entrapped in the base prior to laying the wearing course, some of which remained in the sand cushion course throughout testing. Prior to plane testing, tests on the clay subgrade of this item indicated an average CBR of 5%, a dry density of 92 lb per cu ft, and a moisture content of 30%. On completion of plane testing, the CBR of the subgrade had decreased from 2% to 3%, and the dry density had increased to 95 lb per cu ft, whereas the moisture content remained about the same. The early failure of this item was probably due (a) to the absence of load-spreading characteristics of this base which was composed of a single layer of stone as previously described, and (b) to an inadequate base and pavement thickness for a subgrade having such a low bearing value. In general, it is believed that this type of base allows load transfer in a columnar manner

directly to the subgrade with little chance for load distribution. The presence of free water in the sand cushion may have had some influence.

The sand-clay item also did not satisfactorily withstand the traffic imposed by these tests. Ruts began developing in the traffic lanes during the early stages of testing and became progressively worse but at a slower rate than in the telford item. Ruts developed which allowed the ponding of water and subsequent infiltration into the base around the deflection gage sleeves and through cracks in the 1-in. asphaltic pavement. In general, rutting in this item amounted to from about 2 in. to 3-in. in all traffic lanes on completion of all plane testing. Slight humps appeared generally along the edges of the traffic lanes. By reference to Table 6, it may be noted that surface deflections obtained in this item were greater than those obtained in the thin macadam item, but subgrade deflections were less than those found under the thin macadam base. Observations made in trenches excavated after traffic was completed indicated that progressive shear deformation of the base occurred during traffic. The sand-clay base had a CBR of 35%, a dry density of 122 lb per cu ft, and a moisture content of 10% prior to plane testing. After plane testing, this base had a CBR of 12%, a dry density of 127 lb per cu ft, and a moisture content of 11%. During the testing period the moisture content increased only slightly, the density increased appreciably, and the CBR dropped considerably. The CBR curves developed by the laboratory for this material show that optimum moisture decreases with increase of compactive effort. The final moisture content of 11% was well on the wet side of the optimum moisture for the increased compactive effort. Tests show that for such a condition a lowering of the CBR can be expected. This condition is probably associated with pore pressure development. Thus, even though the moisture content was optimum for the constructed density, the remolding action of this material under traffic with increasing density placed the material above optimum moisture for the increased compactive effort with a resultant loss in CBR value. The clay subgrade in this item had a CBR of 5%, a dry density of 92 lb per cu ft, and a moisture content of 31% prior to plane testing. After plane testing, the subgrade CBR was 3%; the dry density, 94 lb per cu ft; and the moisture content, 30%.

The sand-asphalt item satisfactorily withstood all imposed traffic. Deflection data were not obtained for this item, but pressure data were obtained in the sand base. These pressure data agree favorably with computed pressures and appear to be valid and useful. No rutting or any other detrimental factors, such as cracking or signs of pavement movement, occurred in any part of this item. The sand subgrade appeared to be adequate for the applied wheel loads although the CBR was lower than that required by the CBR design for a 6-in. thickness. Since a very limited number of coverages was applied at a time when the pavement was cold and therefore very strong, no conclusion is believed warranted as to the adequacy of this pavement. Prior to and after plane testing, the sand subgrade had a CBR of 24%, a moisture content of 10%, and a dry density of 110 lb per cu ft. Consolidation of the sand subgrade was noted only to a very minor extent at the surface.

It is believed that the deflection and the pressure data presented in Tables 6 and 7 furnish an adequate basis for the fulfillment of the basic purpose of these tests which was to

\* \* \* furnish data whereby design curves, similar to the California method tentative design curves in Chapter XX [later Chapter 2, Part XII] of the *Engineering Manual* [for War Department Construction<sup>1</sup>], could be developed for a dual wheel load of 60,000 lbs."

Thus, it was not considered necessary to conduct accelerated traffic tests on this test section.

#### CONCLUSIONS

The following conclusions are considered applicable to the data discussed in this paper:

- a. The deflection and the pressure data obtained from the three macadam base items and the pressure data obtained from the sand-asphalt item are believed to be reliable and are considered valid for use in detailed analyses.
- b. An 18-in. depth of telford base and pavement as constructed for these tests, and having a subgrade with a CBR of 5% or less, is unsatisfactory for even a few operations of B-24 or B-29 planes.
- c. The sand-clay base as constructed for these tests is unsatisfactory for B-24 or B-29 plane operations. The base was compacted to optimum conditions practically obtainable by ordinary construction methods. The plane traffic caused additional compaction, and the moisture content of the material increased during plane testing. Laboratory tests show that under these conditions the bearing value of this material decreases.

## OTHER ACCELERATED TRAFFIC TESTS

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In addition to the three major test sections described in previous papers of this Symposium, several accelerated traffic tests were conducted on existing flexible airfield pavements or on specially constructed test sections to furnish data related to the CBR method of design. The tests were conducted at the following locations:

Corpus Christi Municipal Airport, Corpus Christi, Tex.  
Dothan Municipal Airport, Dothan, Ala.  
Fargo Municipal Airport, Fargo, N. Dak.  
Grenier Field, Manchester, N. H.  
Natchitoches Municipal Airport, Natchitoches, La.  
Eglin Field, Valparaiso, Fla.  
Langley Field, Virginia  
Waterways Experiment Station, Vicksburg, Miss.

## DESCRIPTION OF TEST SECTIONS AND TEST PROCEDURES

Table 8 shows the design of the various sections, the traffic imposed, the behavior, and the test data before and after traffic for the various sections. Four of the test sections (Corpus Christi, Dothan, Natchitoches, and Waterways Experiment Station) had regular cross sections with uniform thicknesses, as shown in Table 8. The remaining four test sections were of special design with nonuniform thicknesses. To clarify the design of these sections, the data in Table 8 are supplemented by the longitudinal profiles shown in Fig. 28. A brief description of the four sections with nonuniform thicknesses is given in the following paragraphs. A description of the Waterways Experiment Station test section is also given, since it involved special design considerations.

*Fargo.*—The tests at Fargo Municipal Airport were conducted on a runway before the asphalt wearing course had been placed. The surfacing at the time of the tests consisted of 6 in. of soil cement and was underlain by an imported sandy soil which had a thickness at the edge of 12 in. and a thickness at the center line of 24 in. The subgrade was a highly plastic clay.

*Grenier.*—The test at Grenier Field was conducted on a specially constructed test section 150 ft long and 75 ft wide which was divided longitudinally into five areas having different thicknesses of base course. The wearing surface consisted of a treatment of tar prime, an application of sand, an application of tar, and a cover aggregate of either "pea gravel" or sand.

*Eglin.*—The tests at Eglin Field were conducted on a specially constructed test section with three traffic lanes as follows: Lane 1, 37,000-lb wheel load; lane 2, 50,000-lb wheel load; and lane 3, 15,000-lb wheel load. The thicknesses of wearing course and base course varied as shown in Fig. 28.

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*Langley.*—The tests at Langley Field were conducted on a specially constructed test section. The first test was performed with a 20,000-lb wheel load, and when this did not produce the desired subgrade shear failures a second test was made using a 50,000-lb wheel load. The thickness of base course varied as shown in Fig. 28.

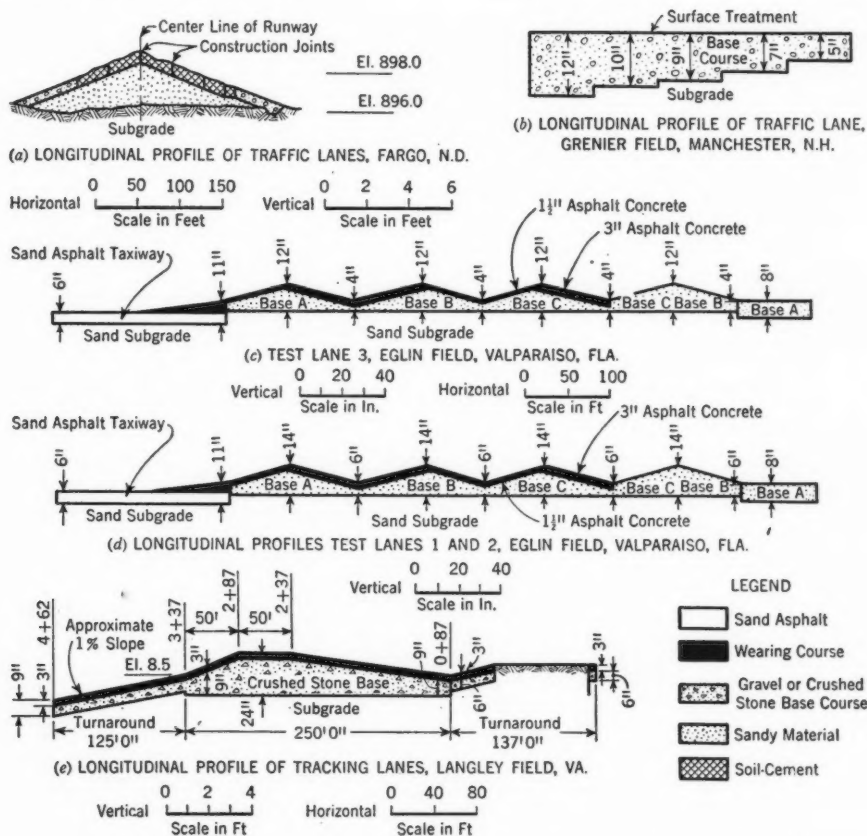


FIG. 28.—LONGITUDINAL PROFILES; FARGO, GRENIER, EGLIN, AND LANGLEY FIELDS

*Waterways Experiment Station.*—The tests at the Waterways Experiment Station were conducted on a specially constructed test section with wheel loads of 15,000 lb and 37,000 lb on single tires and of 60,000 lb on a set of dual tires. This test section was primarily constructed for a study of asphalt wearing course mixes, but several types of base course material were used in order to cover a range of types. The thicknesses of base and wearing course were designed by the CBR method; however 20% additional thickness of base was provided in the main test tracks (items 1-162) to insure adequate subgrade protection since this test section was built primarily to study asphaltic wearing courses.

The accelerated traffic tests were conducted using loaded earth-moving equipment traveling along designated lanes on the pavements. The tests were

TABLE 8.—SUMMARY OF

No.	Name of field	(a) CLASSIFICATION AND THICKNESS DATA								
		Pavement		Base <sup>b</sup>				Subgrade <sup>b</sup>		
		Type <sup>a</sup>	Thickness, in inches	Classi- fication <sup>c</sup>	Thickness, in inches	w <sub>L</sub>	I <sub>P</sub>	Classi- fication <sup>c</sup>	w <sub>L</sub>	I <sub>P</sub>
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	Corpus Christi, Tex.	ac	5	SF	10	30	11	CH	64	38
2	Dothan, Ala.	ac	2	SF	7	35	18	CL	29	12
3	Dothan, Ala.	ac	2	SF	7	35	18	CL	29	12
4	Fargo, N. Dak.	sc	6	SF	12 to 24 <sup>d</sup>	28	13	CH	77	52
5	Fargo, N. Dak.	sc	6	SF	12 to 24	28	13	CH	77	52
6	Fargo, N. Dak.	sc	6	SF	12 to 24	28	13	CH	77	52
7	Grenier, N. H.	st	2	GW	5	22	NP	SW	46	28
8	Natchitoches, La.	ac	2	GF	10	22	6	CH	46	28
9	Eglin, Fla.	ac	1.5 to 3 <sup>i</sup>	SF	1-10.5 <sup>e</sup>	22	3	SP	.....	NP
10	Eglin, Fla.	ac	1.5 to 3 <sup>i</sup>	SF	1-10.5 <sup>e</sup>	20	3	SP	.....	NP
11	Eglin, Fla.	ac	1.5 to 3 <sup>i</sup>	SF	1-10.5 <sup>e</sup>	22	3	SP	.....	NP
12	Eglin, Fla.	ac	1.5 to 3 <sup>i</sup>	SF	3-12.5 <sup>f</sup>	22	3	SP	.....	NP
13	Eglin, Fla.	ac	1.5 to 3 <sup>i</sup>	SF	3-12.5 <sup>f</sup>	20	3	SP	.....	NP
14	Eglin, Fla.	ac	1.5 to 3 <sup>i</sup>	SF	3-12.5 <sup>f</sup>	22	3	SP	.....	NP
15	Eglin, Fla.	ac	1.5 to 3 <sup>i</sup>	SF	3-12.5 <sup>f</sup>	22	3	SP	.....	NP
16	Eglin, Fla.	ac	1.5 to 3 <sup>i</sup>	SF	3-12.5 <sup>f</sup>	20	3	SP	.....	NP
17	Eglin, Fla.	ac	1.5 to 3 <sup>i</sup>	SF	3-12.5 <sup>f</sup>	22	3	SP	.....	NP
18	Eglin, Fla.	sa	5	None	.....	.....	.....	SP	.....	NP
19	Eglin, Fla.	sa	6	None	.....	.....	.....	SP	.....	NP
20	Eglin, Fla.	sa	6	None	.....	.....	.....	SP	.....	NP
21	Langley, Va.	ac	3	GW	12	.....	.....	.....	22	4
22	Langley, Va.	ac	3	GW	21	.....	.....	.....	22	4
23	Item 14.	ac	3	GW	6	.....	NP	CL	39	14
24	Item 14.	ac	3	GW	10	.....	NP	CL	39	14
25	Item 14.	ac	3	GW	13	.....	NP	CL	39	14
26	Item 95.	ac	3	SF	6	22	7	CL	39	14
27	Item 95.	ac	3	SF	10	22	7	CL	39	14
28	Item 95.	ac	3	SF	13	22	7	CL	39	14
29	Item 122.	ac	3	SP	6	.....	NP	CL	39	14
30	Item 122.	ac	3	SP	10	.....	NP	CL	39	14
31	Item 122.	ac	3	SP	13	.....	NP	CL	39	14
32	Item 127.	ac	1.5	SP	7.5	.....	NP	CL	39	14
33	Item 127.	ac	1.5	SP	14.5	.....	NP	CL	39	14
34	Item 160.	ac	6	SP	10	.....	NP	CL	39	14
35	Item 302.	sa	2	GW	9	28	4	CL	39	14
36	Item 302.	sa	2	GW	9	.....	NP	CL	39	14
37	Item 300.	sa	2	GF	9	29	9	CL	39	14
38	Item 300.	sa	2	GF	9	27	10	CL	39	14
39	Item 1.	ac	1.5	GW	11.5	.....	NP	CL	39	14
40	Item 1.	ac	1.5	GW	11.5	.....	NP	CL	39	14

<sup>a</sup> Abbreviations in Col. 2 are defined as follows: ac is asphaltic concrete; sc is soil cement; st is surface treatment; NP means "nonplastic." <sup>b</sup> According to Arthur Casagrande, M. ASCE, *Proceedings, ASCE*, June, 1947, p. 793, expressed as percentages of the modified AASHTO (American Association of State Highway Officials) density, subgrade consolidation. <sup>c</sup> A double wedge-shaped base course; see Fig. 28. <sup>d</sup> To a depth of 24 in. through <sup>e</sup> Also subgrade shear.

continued until failure occurred or a specific number of coverages was completed. Visual observations were made of pavement behavior during the tests and measurements of deformation of the pavement surface both during and after the traffic tests were taken. Field and laboratory tests were performed on the materials in the base and the subgrade after traffic had been stopped. In most cases these tests were made both outside and inside the traffic lanes to represent conditions before and after traffic. Soil classifications are based on the system devised by Professor Casagrande<sup>7</sup> and density relations are compared to the AASHTO classification<sup>25</sup> as modified by the Corps of Engineers.

## DATA FROM TEST SECTIONS

(b) TRAFFIC DATA												Adequate protection <sup>c</sup>	No.
Performance	Wheel load, in pounds	Coverages	Base <sup>d</sup>				Subgrade						
			Before Traffic		After Traffic		Before Traffic		After Traffic				
			Density	CBR	Density	CBR	Density	CBR	Density	CBR			
(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	
38	Base shear	15,000	285	99	23	....	....	96	13	....	....	Yes	1
12	Base shear and	20,000	487	89	10	....	....	90	25	....	....	Yes	2
12	consolidation/	50,000	1,166	89	10	....	....	90	25	....	....	Yes	3
52	Base and subgrade	30,000	570	89	9	....	....	83	3	....	....	No	4
52	shear	20,000	530	89	9	....	....	83	3	....	....	No	5
52	Subgrade shear <sup>a</sup>	12,500	1,470	89	9	....	....	83	3	....	....	No	6
NP	Base consolidation/	15,000	3,710	104	....	109	....	95	35	100	....	Yes	7
NP	Subgrade shear <sup>f</sup>	15,000	582	86	35	86	30	87	5	93	5	No	8
NP		15,000	3,500	95	57	94	53	100	34	100	35	Yes	9
NP		15,000	3,500	91	58	93	95	100	....	101	....	Yes	10
NP		15,000	3,500	94	45	93	61	99	....	99	....	Yes	11
NP	Base and subgrade	37,000	3,500	96	48	97	71	98	22	98	45	Yes	12
NP	consolidation	37,000	3,500	94	45	97	89	94	34	95	42	Yes	13
NP		37,000	3,500	96	28	98	61	99	23	96	31	Yes	14
NP		50,000	3,500	....	....	94	77	....	....	97	45	Yes	15
NP		50,000	3,500	96	38	96	76	94	43	96	41	Yes	16
NP		50,000	3,500	96	38	100	55	98	32	99	26	Yes	17
NP		15,000	3,500	....	....	....	....	94	29	95	40	Yes	18
NP		37,000	3,500	....	....	....	....	95	25	100	14	Yes	19
NP	Subgrade	50,000	3,500	....	....	....	....	....	100	25	....	Yes	20
NP	consolidation	20,000	6,667	....	....	....	....	88	15	94	30	Yes	21
4		50,000	2,733	....	....	....	....	88	15	94	30	Yes	22
14		15,000	3,500	....	81	91	78	....	....	84	28	Yes	23
14	Satisfactory	37,000	1,500	....	81	103	86	....	....	93	31	Yes	24
14		30,000	1,500	....	81	....	87	....	....	98	35	Yes	25
14		15,000	3,500	....	28	99	82	....	....	97	40	Yes	26
14		37,000	1,500	....	28	94	49	....	....	97	34	Yes	27
14	Base consolidation	30,000	1,500	....	28	85	40	....	....	88	43	Yes	28
14		15,000	3,500	....	33	102	51	....	24	95	33	Yes	29
14		37,000	1,500	....	33	100	81	....	24	97	37	Yes	30
14		30,000	1,500	....	33	99	89	....	24	95	31	Yes	31
14	Base and subgrade	15,000	3,500	....	35	102	44	....	21	94	29	Yes	32
14	consolidation	30,000	666	....	35	102	57	....	21	97	41	Yes	33
14		30,000	666	....	34	99	94	....	20	92	30	Yes	34
14	Satisfactory	37,000	100	....	54	92	50	....	38	91	33	Yes	35
14	Subgrade shear	37,000	100	....	54	85	32	....	38	92	14	No	36
14	Satisfactory	30,000	100	....	54	95	44	....	38	97	27	Yes	37
14	Base shear <sup>f</sup>	30,000	100	....	54	92	13	....	38	93	10	No	38
14	Satisfactory	37,000	400	....	....	....	72	....	18	91	18	Yes	39
14	Subgrade shear	37,000	400	....	....	....	56	....	18	90	10	No	40

and  $s_a$  is sand asphalt. <sup>b</sup> In Cols. 6 and 9,  $w_L$  is the liquid limit; and, in Cols. 7 and 10,  $I_P$  is the plasticity index. <sup>c</sup> All density and CBR values are the results of "in-place" tests, and are percentages. The density values are <sup>d</sup> Adequate protection afforded to the subgrade by the thickness of the base and the wearing course. <sup>e</sup> Also pavement and base. <sup>f</sup> Two different thicknesses of ac on a double wedge-shaped base course, see Fig. 28.

## TEST RESULTS AND DISCUSSION

In the tests at Corpus Christi, deformation of the pavement surface of about 1 in. in the traffic lane and a bulge of about 0.5 in. on either side indicated that the base course underwent a small amount of shear deformation and possibly a little consolidation. There was considerable shear deformation or consolidation of the base at only one localized area in which the deformations were about 4 in., and this was attributed to excessive moisture in the base and probably insufficient compaction.

In the test at Dothan, large deformations occurred in the 20,000-lb lane and considerably larger deformations in the 50,000-lb lane. Relatively small upheavals at the sides of the traffic lanes indicated distress which later observations showed to be the result of some shear deformation in the base. The large deformations in the traffic lanes were found to be the results of consolidation in the base and the subgrade.

At Fargo, considerable deformations in the traffic lanes and upheavals at the sides of the lanes occurred throughout the 30,000-lb and the 20,000-lb lanes and to a point in the 12,500-lb lane where total thickness of the soil cement and the underlying sandy soil was 24 in. From observations, it was concluded that shear deformations had taken place in both the sandy base material and the subgrade throughout the 20,000-lb and the 30,000-lb lanes. No shear deformation occurred in either the sandy base or the subgrade in the 12,500-lb lane where the total thickness above the subgrade was more than 24 in.

No in-place CBR values were determined on the base material at Grenier Field either before or after traffic, but the average laboratory remolded and soaked value for this material was 75%. Traffic produced no signs of shear deformation in either the base or the subgrade, but there was evidence of consolidation in the base. The surface treatment was inadequate to withstand the abrasive action of traffic.

In the test at Natchitoches, large deformations occurred in the tracking lanes with adjacent upheavals. The upheavals indicated that subgrade and possibly base shear deformation occurred. However, the upheavals were not large enough to compensate for the large deformations inside the tracking lanes, and it was concluded that subgrade consolidation also occurred.

No shear deformation occurred in the base or subgrade in any traffic lane of the Egin Field test section. A thickness of base and pavement of 4 in. which was the minimum in the test section was adequate to prevent shear deformation in the subgrade under the 15,000-lb wheel load. A thickness of base and pavement of 6 in. was sufficient to prevent shear deformation in the subgrade for the 37,000-lb and the 60,000-lb wheel loads. The 6-in. thickness was the minimum in these lanes. Some consolidation occurred in all base and subgrade materials under each type of wearing course in each traffic lane. The maximum deformations were 5.3 in., 5.5 in., and 2.2 in. for lanes 1, 2, and 3, respectively. All wearing courses were adequate in all traffic lanes.

In the tests at Langley Field, considerable settlement had occurred in the 20,000-lb lane at the end of the test, which had a minimum total thickness of base and pavement of 12 in. This settlement was all due to consolidation, except in the turnarounds where subgrade shear occurred. The subgrade shear occurred under the 20,000-lb wheel load where the total thickness was only 9 in. Very large deformations had occurred in the 50,000-lb lane at the end of the test. These were partly caused by consolidation, but subgrade shear deformation also occurred where the total thickness of base and pavement was 22 in. or less. Excessive springing occurred in the tests, which indicates that excess hydrostatic pressure had developed in the pore water. The subgrade CBR at the start of the test was 15% and was 30% after the tests were completed. Analysis is difficult, since the CBR at the time the springing occurred

is not known. It is considered that thicknesses of 12 in. and 24 in. are considerably more than adequate to prevent shear deformation in a subgrade with a CBR of 30 under 20,000-lb and 50,000-lb wheel loads, respectively, provided no hydrostatic excess pressure occurs in the pore water.

In the tests at the Waterways Experiment Station, the performance of pavements in items 14, 95, and 122 is shown as adequate for subgrade protection in Table 8, although some consolidation took place in both base and subgrade as evidenced by a slight deformation of the surface and increases in in-place CBR values. Rather large deformations occurred in items 127 and 160. On the basis of observations, it was concluded that they were caused by base and subgrade consolidation, since the deformations were not accompanied by upheavals at the sides of the tracking lanes, and since in-place CBR values of both base and subgrade increased. In items 1 and 302, subgrade shear occurred, which was evidenced by upheavals. In item 300, subgrade and base shear occurred, which was also evidenced by upheavals outside the trafficked area.

#### CONCLUSIONS

Since the data presented in this paper are to be used with other data in a summary analysis of the CBR method of design in a later paper, a few general but no specific conclusions will be made here. The following conclusions are based on the data shown in Table 8:

- a. Shear failure did not occur in nonplastic to slightly plastic base or subgrade material;
- b. Shear failures in either base or subgrade developed at a rapid rate once the condition for failure existed;
- c. In general, nonplastic base courses were consolidated to more than 100% modified AASHO density by traffic, and bases having some plasticity were consolidated to more than 95% of modified AASHO density; and
- d. All nonplastic subgrades were consolidated to more than 95% of modified AASHO density by traffic and, in general, plastic subgrades were consolidated to more than 90%.

## DESIGN CURVES FOR SINGLE WHEEL LOADS

BY C. R. FOSTER,<sup>32</sup> ASSOC. M. ASCE

As explained in a previous paper, the CBR design curves for heavy wheel loads were extrapolated from highway data in the initial stages of the airfield construction program. Subsequently, the curves were correlated with the service behavior of pavements under both accelerated traffic and actual airplane traffic. The purpose of this paper is to present the data that have been accumulated on this subject and to show the correlations that have been made. A comparison between the data and the present CBR thickness design curves is presented.

The general method of correlating the curves with service behavior has been to select satisfactory pavements and pavements that failed and to compare the CBR of the subgrade, or layer being studied, and the total thickness of overlying base and pavement with the CBR thickness design curves. All comparisons are based on "in-place" CBR values which are considered to represent the condition of the soil at the time traffic is applied.

Pavements included in the "failed" category are limited to those in which shear deformation occurred in a subgrade or in a lower base course overlain by a high-quality base in which no shear deformation occurred. In no instance is information used where there was evidence of shear deformation in the base. For instance, data from items 1 and 2 in the Barksdale test (see Table 4(c)) are not used because shear deformation occurred in the base as well as in the underlying subgrade. The data from item 5 at Barksdale, where shear deformation occurred in the subgrade but not in the base, are used (see Table 9(a), line 49, and Table 9(b), line 25).

Certain criteria have been used in selecting pavements from the standpoint of intensity of traffic. A study of the available records where failure occurred showed that where the failure is caused by shear deformation it usually occurs after a relatively few repetitions. In approximately fifty cases for which reliable data are available, 96% of the failures occurred before 2,000 coverages had been applied. This amount of traffic is relatively small if the total that can occur at an airfield is considered. Traffic counts have shown that approximately 75% of the traffic on a 150-ft runway occurs in the central 50 ft. Assuming traffic in the central 50 ft to be uniformly distributed, a 30,000-lb wheel load with two tire prints 1.33 ft wide will produce a coverage in approximately nine landings and take-offs in the central 50 ft which would require 12 cycles of operation on the field. One landing and one take-off constitute a cycle. A reasonable value for the maximum number of cycles of operation per day for planes of this weight during the pilot training period was 270, which would produce slightly more than 20 coverages per day in the central 50 ft of the runway. On this basis, 2,000 coverages represent about one hundred days of operation.

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The failures under consideration are those resulting from shear deformation caused by overstress. Failures due to consolidation and to surface deterioration occur over a much longer period. There are also cases of failure from saturation of the subgrade or base that occur after several years of operation. However, when conditions become critical so that the subgrade is overstressed, failure occurs after a relatively few additional repetitions.

On the basis of the foregoing considerations, airfield pavements that have been subjected to more than 30,000 cycles of operation (approximately 2,000 coverages) without showing distress are considered satisfactory. Accelerated traffic test pavements that have successfully withstood 2,000 coverages are also considered adequate. No minimum traffic criteria have been applied to pavements that failed.

Table 9 shows the pertinent data that have been accumulated from the various studies. All pavements for which reliable traffic and CBR data are available are shown. Table 9 includes the data obtained from the accelerated traffic tests described in previous papers of this Symposium together with the information available from actual airfields. It gives the name of the field and facility; the identification of test area; the wheel load; the amount of traffic at the time of test; the type and thickness of wearing course, base, and subgrade; and the CBR of the various layers. The data are presented in two groups to facilitate plotting. One group gives data for wheel loads less than 30,000 lb; the other, for wheel loads of 30,000 lb and greater. Each line in the two groups is given a number for reference on the plots.

Fig. 29(a) shows CBR design curves for wheel loads of 15,000 lb, 20,000 lb, 25,000 lb, 30,000 lb, 40,000 lb, 50,000 lb, and 60,000 lb. The 15,000-lb, 40,000-lb, and 60,000-lb curves are the same as the runway curves presented in the *Engineering Manual for War Department Construction*<sup>23</sup> prepared by the Office of the Chief of Engineers. The others are interpolations from curves given in the manual. The interpolated curves are presented to facilitate comparison with the field data. These design curves represent a slight modification from the original extrapolations as presented in Symposium paper No. 3 in that the thickness requirements have been reduced slightly for the higher CBR values in the range of cohesionless materials. This modification was made on the basis of the results of accelerated traffic tests. Also, minimum thicknesses have been established for the various wheel loads. The minimum thicknesses are specified to insure satisfactory performance from settlement, weather, and other features, as well as to prevent shear in the subgrade. For comparison, the California design curves representing light and heavy highway experience used in the extrapolations are shown.

Fig. 29(a) is a plot of data for wheel loads less than 30,000 lb; Fig. 29(b) is a plot of data for wheel loads of 30,000 lb and more. The data from the accelerated traffic tests and the airfield pavements are identified by appropriate symbols. Variable thicknesses were usually tested in the accelerated traffic tests; therefore, three symbols were used for the data. Where it was possible to determine a thickness of base and pavement adequate to prevent shear deform-

<sup>23</sup> "Airfield Pavement Design: Flexible Pavements," *Engineering Manual for War Department Construction*, Office of Chf. of Engrs., U. S. Army, Washington, D. C., July, 1946, Pt. XII, p. 11, Fig. 2.

TABLE 9.—CBR DATA FOR THE DEVELOPMENT OF DESIGN CURVES FOR SINGLE WHEEL LOADS

Line	Army airfield	Facility <sup>a</sup>	Pit No.	SUBGRADE OR SUBBASE <sup>b</sup>				OVERLYING BASE <sup>b</sup>				SURFACE		Total thickness, in inches	TRAFFIC		Performance satisfactory <sup>d</sup>	
				Type <sup>c</sup>	wL (%)	IP (%)	CBR (%)	Type <sup>c</sup>	wL (%)	IP (%)	CBR (%)	Thickness, in inches	Type <sup>c</sup>		Wheel load, kips	Coverages		
(1)	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
(a) WHEEL LOADS LESS THAN 30,000 LB																		
1	Fargo, N. Dak.	Test section	.....	CH	79	55	3	SF	28	13	9	18.0-24.0	sc	6.0	12.0-30.0	12.5	1,470	Yes <sup>e</sup>
2	Florence, S. C.	NE-SW	.....	SP	.....	.....	13	la/	.....	.....	.....	11.5	ac	2.5	14.0	13.0	3,134	Yes
3	Pan-American <sup>g</sup>	E-W	.....	SC	.....	.....	38	hr <sup>c</sup>	.....	.....	.....	6.0	st	0.5	6.5	13.0	4,051	Yes
4		NW-SE1	.....	CH	70	44	10	la	NP-17	1	108	10.0	ac	2.0	12.0	15.0	11,258	Yes
5		NW-SE1	.....	CH	70	44	6	la	NP-17	NP-1	27-108	15.0	ac	2.0	17.0	15.0	11,258	Yes
6		NW-SE1	.....	CH	70	44	37	GW	17	1	150	10.0	ac	2.0	17.0	15.0	11,258	No <sup>a</sup>
7	Bergstrom Field, Austin, Tex.	NW-SE1	.....	CH	70	44	11	la	NP-17	1	150	10.0	ac	2.0	12.0	15.0	11,258	Yes
8		NW-SE1	.....	CH	70	44	30	GW	17	1	135	10.0	ac	2.0	17.0	15.0	11,258	Yes
9		NW-SE1	.....	CH	70	44	12	la	NP-17	1	30-135	15.0	ac	2.0	17.0	15.0	11,258	Yes
10		T-1	.....	ML	22	39	19	la	17-22	3	150+	15.0	ac	2.0	12.0	15.0	23,700	Yes
11		West N-S	.....	CH	58	39	10	la	17-22	3-7	19-150+	15.0	ac	2.0	17.0	15.0	1,741	Yes
12		West N-S	.....	CH	71	37	23	GP	16	2	150+	9.0	ac	3.0	12.0	15.0	1,741	Yes
13	Berry Field, Nashville, Tenn.	West N-S	.....	ML	37	11	43	GP	16	2	124	9.0	ac	3.0	12.0	15.0	1,741	Yes
14		West N-S	.....	CL	41	20	27	GP	16	2	139	9.0	ac	3.0	12.0	15.0	1,741	Yes
15		West N-S	.....	CH	51	29	16	GP	16	2	93	9.0	ac	3.0	12.0	15.0	1,741	Yes
16		T-4A	.....	CL	28	13	37	GF	25	9	88	8.5	ac	3.5	12.0	15.0	8,888	Yes
17	Dodge City (Kans.) airfield	T-4A	.....	CL	47	32	14	la	25-28	9-13	37-88	15.5	ac	3.5	19.0	15.0	8,888	Yes
18		Test section	.....	SP	NP	NP	35	SF	21	3	60	1.0-10.5	ac	1.5 and 3.0	4.0-12.0	15.0	3,500	Yes <sup>f</sup>
19	Eglin, Fla.	Test section	.....	SW	NP	NP	30	none	NP	NP	.....	5.0-12.0	sa	6.0	6.0	15.0	3,500	Yes <sup>f</sup>
20	Grenier, New Hampshire	Test section	.....	SW	NP	NP	30	GW	NP	NP	.....	5.0-12.0	st	3.0	5.0-12.0	15.0	3,710	Yes <sup>f</sup>
21		T-4	.....	SC	19	1	22	SF	19	3	45	7.0	ac	2.0	10.0	15.0	3,475	Yes
22	Lawson, Fort Benning, Ga.	T-4	.....	SC	19	1	25	SF	19	3	32	8.0	ac	2.0	10.0	15.0	3,475	Yes
23		T-4	.....	CL	39	14	28	GW	.....	NP	78	6.0	ac	3.0	9.0	15.0	3,500	Yes
24	Waterways Experiment Station, Vicksburg, Miss.	Test section	14 <sup>k</sup>	CL	39	14	40	ML	.....	NP	82	6.0	ac	3.0	9.0	15.0	3,500	Yes
25		Test section	95 <sup>k</sup>	CL	39	14	33	SP	.....	NP	51	6.0	ac	3.0	9.0	15.0	3,500	Yes
26		Test section	122 <sup>k</sup>	CL	39	14	33	SP	.....	NP	60	6.0	ac	3.0	9.0	15.0	3,500	Yes
27	Rocky Ford (Colo.) Field; Auxiliary to the La Junta (Colo.) Field	E-W	.....	CL	28	17	18	GW	NP-28	NP-17	18-60	6.0	ac	2.5	8.5	16.0	2,902	Yes
28		E-W	.....	CL	30	20	6	la	NP-28	NP-17	18-60	14.0	ac	2.5	16.5	16.0	2,902	Yes
29		E-W	.....	CL	24	16	42	GW	NP-24	NP-16	42-103	6.0	ac	2.5	8.5	16.0	2,902	Yes
30		E-W	.....	CL	30	18	11	la	NP-24	NP-16	42-103	14.0	ac	2.5	8.5	16.0	2,902	Yes

[illegible]

TABLE 9.—(Continued)

Line	Army airfield	Facility <sup>a</sup>	Pit No.	SUBGRADE OR SUBBASE <sup>b</sup>				OVERLYING BASE <sup>b</sup>				SURFACE		Total thickness, in inches	TRAFFIC		Performance-satisfactory <sup>d</sup>	
				Type <sup>c</sup>	wL (%)	I <sub>P</sub> (%)	CBR (%)	Type <sup>c</sup>	wL (%)	I <sub>P</sub> (%)	CBR (%)	Thickness, in inches	Type <sup>c</sup>		Wheel load, in kips	Coverages		
(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)

(b) WHEEL LOADS OF 30,000 LB AND MORE (for Footnotes, See Table 9(a))

1	Clovis, N. Mex.	N-S <sup>1</sup> B <sup>1</sup>	4	CL	23	9	28	GF	24	7	52	12.0	ac	1.5	13.5	30.0	1,709	Yes
2	Fargo, N. Dak.	Test section	...	SP	28	13	9	ac	22	3	175+	6.0	ac	2.5	6.0	30.0	1,570	No <sup>a</sup>
3		N-S	3	ML	NP	NP	NP	SP	24	6	79	8.5	ac	2.5	11.0	30.0	1,824	Yes
4	Kirtland <sup>m</sup>	N-S	4	ML	NP	NP	NP	SP	24	6	79	8.0	ac	2.0	10.0	30.0	1,824	Yes
5		N-S	6	SP	25	4	47	SP	24	5	62	8.5	ac	2.5	11.0	30.0	7,612	Yes
6		N-S	3	CL	NP	NP	NP	GF	24	3	100+	9.0	ac	2.0	11.5	30.0	7,612	Yes
7		E-W	3	CL	44	24	17	GF	21	3	85+	9.0	ac	2.0	11.0	30.0	2,951	Yes
8		E-W	5	CL	39	20	18	GF	21	4	100+	9.0	ac	2.0	11.0	30.0	2,951	Yes
9	Pueblo, Colo.	E-W	7	CL	39	20	16	GF	21	4	100+	9.0	ac	2.0	11.0	30.0	2,951	Yes
10		T-6	9	CL	35	18	25	GF	18	1	97	7.0	ac	5.0	12.0	30.0	4,716	Yes
11	Waterways Experiment Sta-	Test section	14 <sup>2</sup>	CL	39	14	35	GW	NP	NP	87	13.0	ac	3.0	16.0	30.0	1,500	Yes
12	tion, Vicksburg, Miss.	Test section	95 <sup>2</sup>	CL	39	14	43	ML	NP	7	38	13.0	ac	3.0	16.0	30.0	1,500	Yes
13		N-S	3	SP	NP	NP	NP	SW	NP	NP	46	9.0	ac	4.0	13.0	30.0	48,083	Yes
14	Yuma, Ariz.	N-S	4	SP	NP	NP	NP	SW	NP	NP	43	8.0	ac	4.0	12.0	30.0	48,083	Yes
15		N-S	7	SP	NP	NP	NP	GF	NP	NP	62	10.0	ac	4.0	14.0	30.0	48,083	Yes
16		Test section	.....	SP	NP	NP	NP	SP	21	3	60	3.0-12.5	ac	1.5 and 3.0	6.0-14.0	37.0	3,500	Yes <sup>a</sup>
17	Eglin, Fla.	Test section	.....	SP	NP	NP	NP	none	.....	.....	.....	.....	sa	6.0	37.0	3,500	Yes <sup>a</sup>	
18		Test section	14 <sup>2</sup>	CL	39	14	31	GW	NP	NP	86	10.0	ac	3.0	13.0	37.0	1,500	Yes
19		Test section	95 <sup>2</sup>	CL	39	14	34	ML	NP	7	49	10.0	ac	3.0	13.0	37.0	1,500	Yes
20	Waterways Experiment Sta-	Test section	122 <sup>2</sup>	CL	39	14	37	SP	NP	NP	81	10.0	ac	3.0	13.0	37.0	1,500	Yes
21	tion, Vicksburg, Miss.	Test section	1 <sup>2</sup>	CL	39	14	10	GF	NP	NP	56	7.5	ac	1.5	9.0	37.0	400	No <sup>a</sup>
22		Test section	302 <sup>2</sup>	CL	39	14	14	GW	NP	NP	32	9.0	sa	2.0	11.0	37.0	100	No <sup>a</sup>
23		Test section	.....	CH	57	34	5	GF	NP	NP	80	6.0-42.0	ac	3.0	6.0-48.0	40.0	3,730	No <sup>a</sup>
24	Stockton, Calif.	Test section	5 <sup>2</sup>	CL-CH	45	28	5-6	GF	NP	NP	80	23.0-27.0	ac	3.0	24.0-43.0	50.0	5,000	No <sup>a</sup>
25	Barksdale <sup>a</sup>	Test section	.....	SP	NP	NP	NP	SP	21	3	60	3.0-12.5	ac	1.5 and 3.0	6.0-14.0	50.0	3,500	Yes <sup>a</sup>
26	Eglin, Fla.	Test section	.....	SP	NP	NP	NP	SP	21	.....	.....	.....	sa	6.0	6.0	50.0	3,500	Yes
27		Test section	.....	SP	NP	NP	NP	SP	21	.....	.....	.....	sa	6.0	6.0	50.0	3,500	Yes
28	Langley Field, Va.	Test section	.....	SF	22	4	30	GW	NP	NP	80	9.0-21.0	ac	3.0	12.0-24.0	50.0	2,733	Yes <sup>a</sup>

ation in the subgrade, a half black rectangle has been used. Inasmuch as the half black rectangles represent the thickness selected as necessary to prevent shear deformation, the design curves should pass through them. A solid black rectangle indicates that shear deformation occurred under the maximum thickness tested; hence, the symbol indicates that a greater thickness is required.

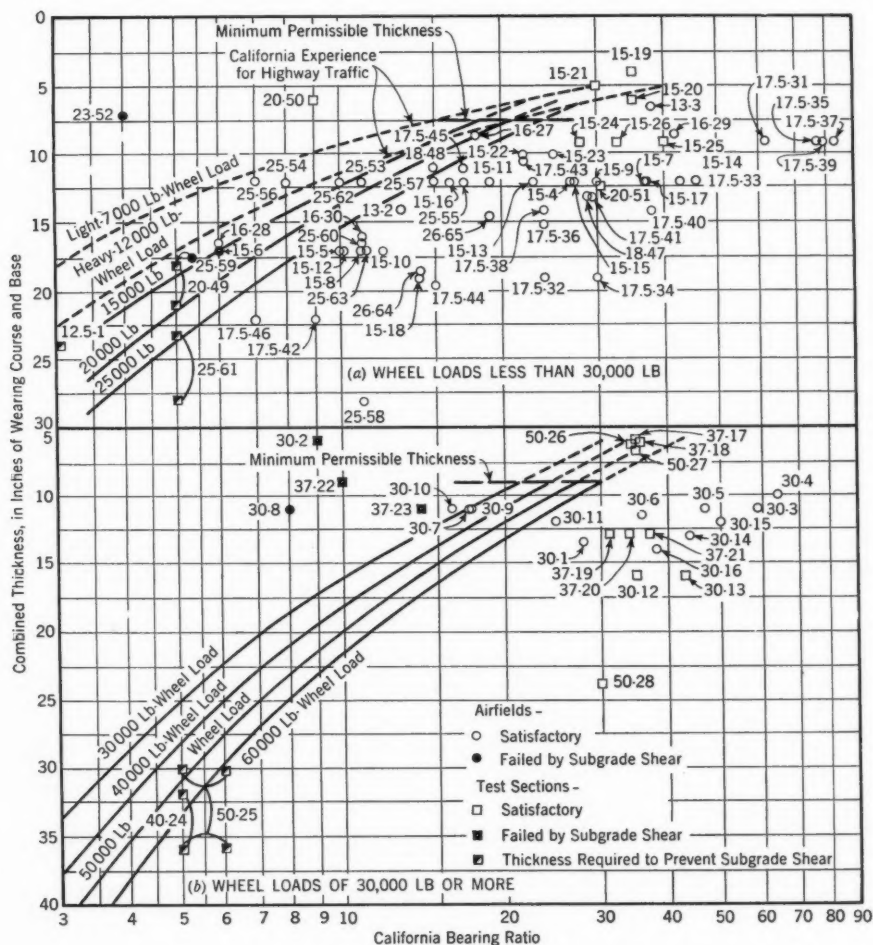


FIG. 29.—CORRELATION OF CBR DESIGN CURVES WITH FIELD DATA

An open rectangle is used where no shear deformation occurred in the subgrade at the minimum thickness of base and pavement tested; hence, a lesser thickness may be adequate. Variable thicknesses were usually not found at the airfields and only two symbols were used for the airfield data. A solid black circle denotes a pavement that failed and an open circle denotes a satisfactory pavement. The first number beside each point is the wheel load, in kips or

thousands of pounds, and the second number refers to the appropriate line in Table 9.

A study of Fig. 29(a) reveals that the data for wheel loads of less than 30,000 lb agree with the general pattern of the design curves. In general, the points representing adequate pavements plot below the respective design curves, which is the proper relationship since the indications are that the thickness was satisfactory and that possibly a lesser thickness may have been satisfactory. Exceptions are three points from the Campbell Army Airfield, Camp Campbell, Kentucky (lines 53, 54, and 56, Table 9(a)), and one point from Woodward Army Airfield, Woodward, Okla. (line 62). Little weight is given the data from Camp Campbell since the CBR values were determined after a 7-month period in which practically no traffic occurred. The point from Woodward (line 62) plots at a depth which is about 3.5 in. less than the respective design curve. Except for the instances noted, the points representing satisfactory pavements plot either very close to, or below, the respective curves. It is also noted that the points representing failures plot above the respective design curves, which is proper since the indications are that more thickness was required. In the case of the failure at Bergstrom Field, Austin, Tex. (line 6, Table 9(a)), where a subgrade with a CBR of 6 failed under a 15,000-lb wheel load, the failure point plots directly on the 15,000-lb curve. Data are available from the Stockton and Barksdale test sections where the thickness required to prevent subgrade shear was bracketed. In the Stockton data (line 61) the bracketed thickness tends to fall below the respective curve, and in the Barksdale data (line 49) the points tend to plot above the respective curve. The data for the heavier wheel loads shown in Fig. 29(b) also are in agreement with the general pattern of the design curves although the data are not so numerous as those for the lighter wheel loads. All the points representing satisfactory pavements fall either on or below the respective design curves and the points representing failures plot above the respective design curve. One point plots fairly close to the curves but the others are well above the curves. This point is for a 37,000-lb wheel load and a CBR of 14 from the Waterways Experiment Station test section (line 23, Table 9(b)). The data from Stockton and Barksdale, where the thickness required to prevent shear in the subgrade was bracketed, plot below the respective design curve. The data from Stockton (line 61) plot well below the curve. In the case of the Barksdale data (line 49) the curve passes through the upper limit of the bracketed thickness.

From a general study of both sets of curves in Fig. 29, certain features can be noted. One is that the curves appear to be conservative in the higher CBR values. Except for the point for the 37,000-lb wheel load from the Waterways Experiment Station test section (line 23, Table 9(b)), there are no failure points which approach the design curves for the higher CBR values. Reductions in the thickness requirements have been made in this range, however; and, as shown in Fig. 29(a), the requirements are less than the original California highway experience. Also, any possible thickness reductions in this range would be small since minimum thicknesses have been established for the various wheel loads for other reasons than the prevention of subgrade shear.

Another feature that is noted is that the curves and data for the lower CBR values are in reasonably good agreement for wheel loads less than 30,000 lb; however, the agreement is not so good for the heavier wheel loads. There is evidence that the thickness requirements should probably be increased about 4 in. or 5 in. for the 50,000-lb and the 60,000-lb wheel loads.

In summary, the data that have been obtained from the accelerated traffic tests follow the general pattern of the design curves. There are indications that the design curves are slightly conservative for the higher CBR values. Any reduction in thickness requirements, however, would be slight, and its application would be limited as minimum thickness requirements prevail for CBR values above the range of from 20 to 30. For the lower CBR values, the design curves appear to be approximately correct for the lighter wheel loads, but there are indications that thickness requirements for the heavier wheel loads may have to be increased 4 in. or 5 in. No change in the present design curves is contemplated, however, until additional data are received.

In the analysis, considerable weight was given to the data from the accelerated traffic tests. From the standpoint of test results, these data are probably the most reliable since the amount of traffic was carefully recorded and all testing was very carefully done. When the first traffic tests were conducted, there was a question of whether the thickness of base and pavement required in the traffic test would be correct for an airfield pavement under heavy traffic. The fact that the data from the accelerated traffic tests are in agreement with the data from airfields is evidence that the thickness requirements in the traffic tests approximate, reasonably well, those for airfield pavements under heavy traffic.

The points plotted in Fig. 29 represent the pertinent data that are available. Although there is some deviation of the field data from the requirements of the design curves, absolute accuracy in this kind of work cannot be expected, and it is considered that the field data and the design curves are in substantial agreement. The curves are believed to represent the total thickness of high-quality base and pavement required to prevent shear deformation in the underlying layer under the maximum intensity of single wheel traffic that occurs on runways at military airfields.

## DESIGN CURVES FOR VERY HEAVY MULTIPLE WHEEL ASSEMBLIES

BY W. K. BOYD,<sup>34</sup> M. ASCE, AND C. R. FOSTER,<sup>32</sup> ASSOC. M. ASCE

In the preceding Symposium papers, data have been presented leading to the development of design criteria for single wheel loads. When the B-29 plane was introduced with dual wheel assemblies, it was necessary to evaluate the effect of the dual wheel in comparison with the single wheel if maximum use of the plane was to be obtained. At the request of the U. S. Army Air Forces, the Corps of Engineers initiated a study to develop design curves for flexible pavement thickness and for other requirements of the B-29 plane.<sup>35</sup> These curves show definitely that a considerable saving in base thickness can be effected by the use of dual wheels. Further study of the problem has evolved procedures whereby the method used in developing the B-29 curves can be extended and applied to any wheel load assembly. Curves for a given assembly can be developed

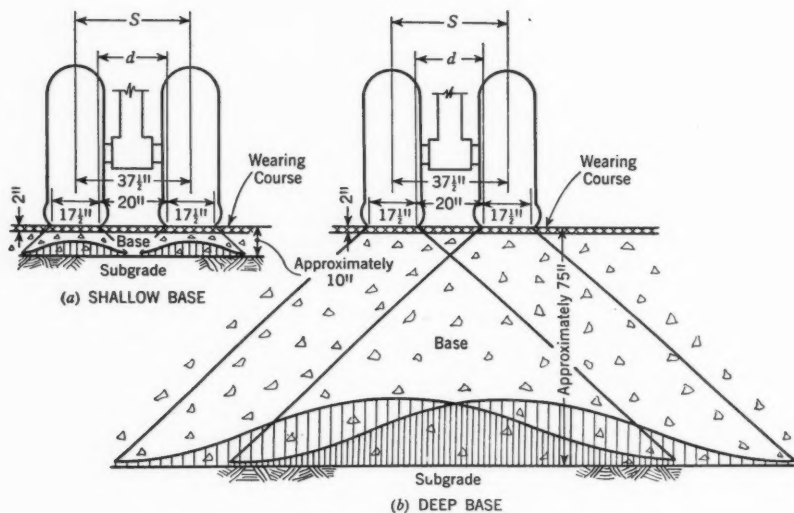


FIG. 30.—SCHEMATIC DIAGRAM OF B-29 DUAL WHEEL ASSEMBLY

readily which will permit the rapid evaluation of the efficiency of variations in the spacing of the wheels. The purpose of this paper is to present the method by which the B-29 curves were developed and to show the extension that may be applied to any given assembly.

Although the method described herein was developed primarily for airplanes, it is also applicable in the determination of the effect of multiple truck

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<sup>35</sup> "Certain Requirements for Flexible Pavement Design for B-29 Planes." U. S. Waterways Experiment Station, Vicksburg, Miss., 1 August 1945.

axles on highway pavements. The method can be used to find axle spacings and loads that will produce no greater detrimental effect than is produced by a given load on a single axle.

Fig. 30 is a schematic diagram of the B-29 dual wheel assembly on a thin flexible pavement and on a thick flexible pavement. The design load on the dual assembly of the B-29 plane is 60,000 lb or 30,000 lb on each tire. Fig. 30(a) refers to the case of a shallow base. In this case the bearing value of the subgrade must necessarily be high in order that the thin base may be adequate to prevent failure in the subgrade. Fig. 30(b) refers to the case of a thick base. The subgrade in this case would be poor, requiring a substantial thickness of base to prevent failure. On both diagrams, shading is shown under each tire which is intended to suggest the distribution of critical stresses induced in the subgrade. The distributions shown represent no particular case and are used only for the purpose of this discussion. Regardless of the accuracy of the distribution as drawn, it must be conceded that at some shallow base thickness the two wheels of the dual assembly will stress the subgrade as practically independent 30,000-lb units, with little or no overlapping of stresses. Under a very

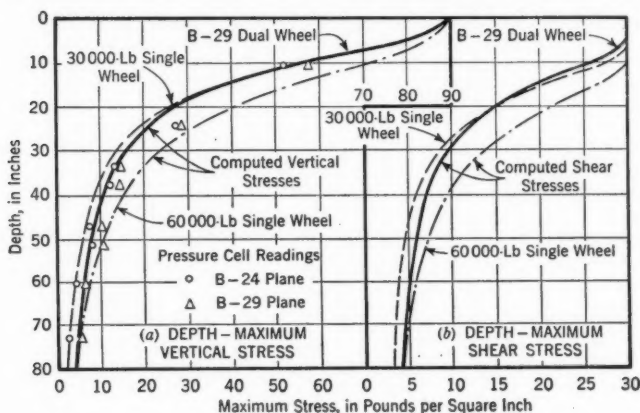


FIG. 31.—VERTICAL STRESSES AND SHEAR STRESSES

deep base (Fig. 30(b)), the stresses from the two wheels overlap considerably. Again, it must be recognized that there is a depth at which the overlapping of stresses would be so great that the stress induced in the subgrade from the dual wheel assembly, for all practical purposes, would be the same as that induced by a single 60,000-lb wheel load. Thus it was reasoned that the thickness design curves for the B-29 plane must range between the curves for the 30,000-lb and the 60,000-lb single wheels. As previously stated, thickness design curves were available for single wheels. The problem of determining design curves for the B-29 dual wheel was narrowed to finding:

- a. The thickness at which each tire stresses the subgrade as an independent unit.
- b. The thickness at which the two tires stress the subgrade as one single unit.

It was further reasoned that, if those two thicknesses could be determined, thickness requirements between the two values would vary in an orderly manner.

The thickness at which each tire of the B-29 dual assembly acts as an independent unit, and the thickness at which the two tires act as a single unit, were determined by comparisons of vertical stresses, shearing stresses, and deflections. These comparisons are illustrated by Figs. 31 to 35. Details of the

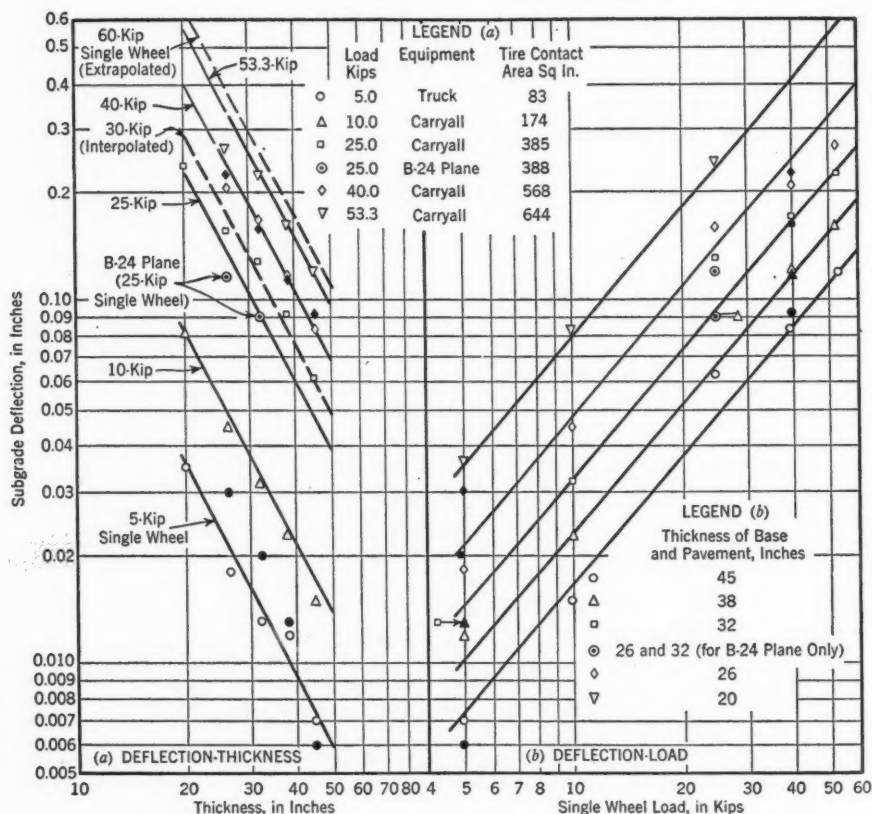


FIG. 32.—SUBGRADE DEFLECTIONS UNDER STANDING LOADS, STOCKTON RUNWAY TEST SECTION

manner in which these data were obtained are not essential for present purposes; however, these details appear elsewhere.<sup>35</sup>

Fig. 31(a) is a plot of the variation of maximum vertical stress (pressure in pounds per square inch) with depth, under a wheel load. Three curves are shown, representing the computed maximum vertical stresses under 30,000-lb single wheels, 60,000-lb dual wheels, and 60,000-lb single wheels, which had the physical characteristics listed in Table 10. Computations were made using Boussinesq's formulas assuming homogeneous material. Stress charts devised

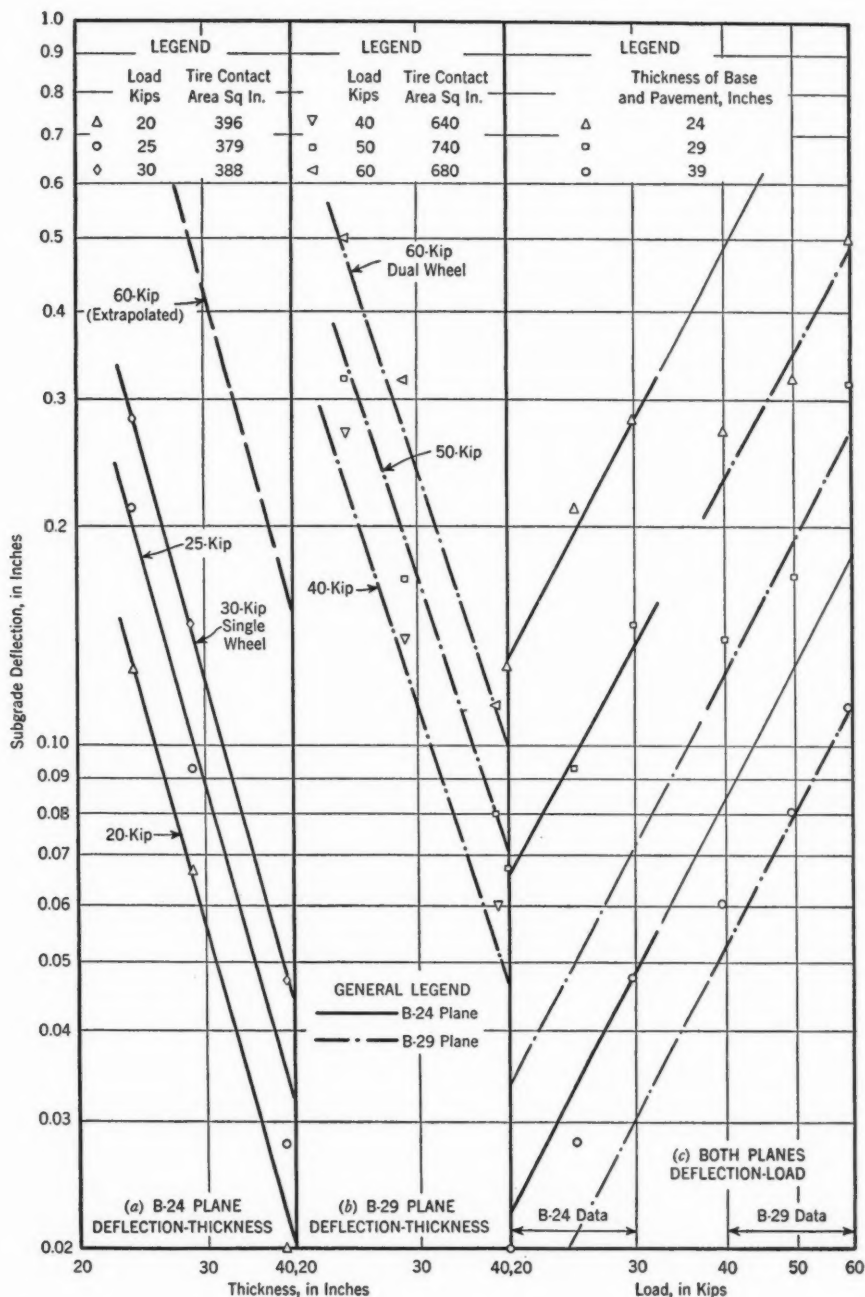


FIG. 33.—SUBGRADE DEFLECTIONS UNDER STANDING LOADS, MARIETTA PAVEMENT BEHAVIOR TESTS

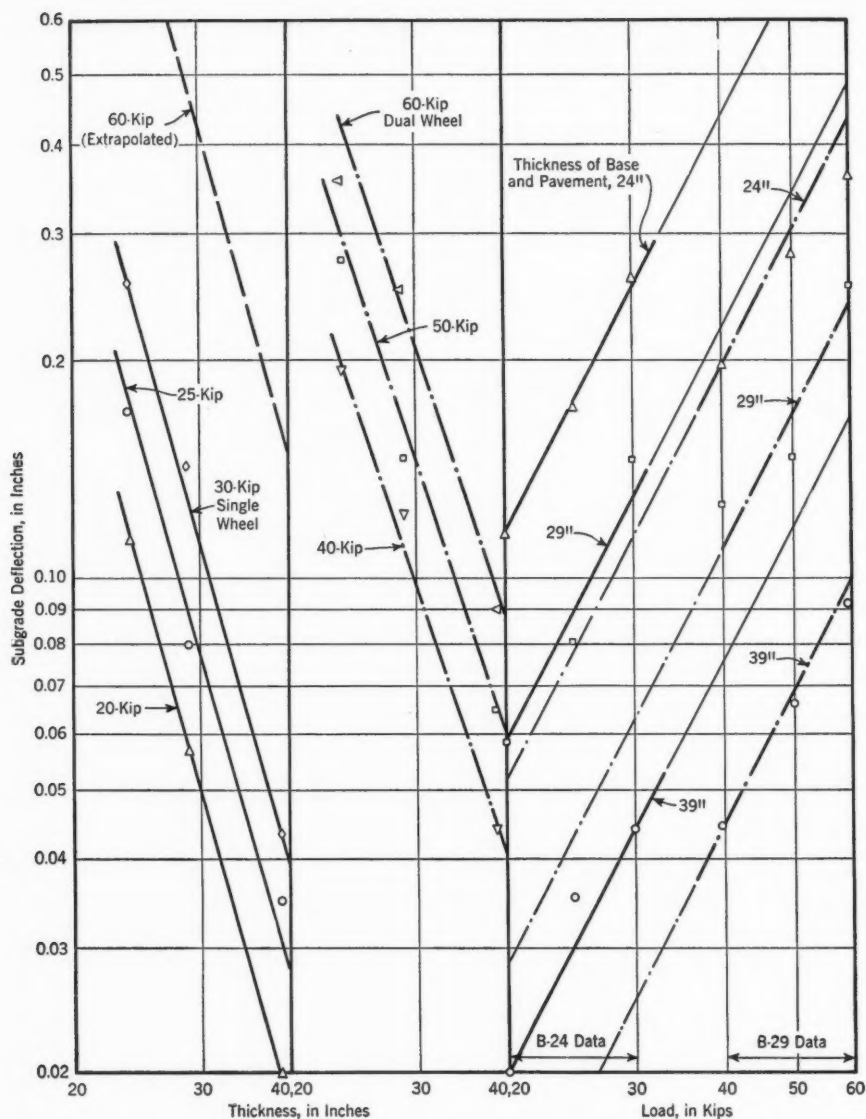


FIG. 34.—SUBGRADE DEFLECTIONS UNDER MOVING LOADS (LEGENDS THE SAME AS FIG. 33)

by Professor Newmark<sup>36</sup> were also used in the computations. As would be expected, the vertical stress for any wheel load is greatest immediately under the tire and decreases with depth; and, at any given depth, the heavier wheel load induces a greater stress. Also shown on the plot are measured observations of vertical stresses obtained from pressure cells under a B-24 plane which

<sup>36</sup> "Influence Charts for Computation of Stresses in Elastic Foundations," by Nathan M. Newmark, *University of Illinois Bulletin*, Vol. 40, No. 112, Urbana, Ill., November 10, 1942.

had a 30,000-lb load on a single wheel, and under a B-29 plane which had a 60,000-lb load on dual wheels. The circles and the light curve on the left, as well as the triangles and the heavy curve, represent comparable loading. The agreement between actual observation and theory in this case is considered reasonably close. Attention is directed to the fact that the theoretical 30,000-lb

TABLE 10.—PHYSICAL CHARACTERISTICS OF WHEEL LOADS

WHEEL				ELLIPTICAL CONTACT SURFACE			
Type	Distance center to center (in.)	Plane	Load (lb)	Major axis (in.)	Minor axis (in.)	Area (sq in.)	Pressure (lb per sq in.)
Single.....	37.0	B-24	30,000	27.12	15.73	335	89.55
Dual.....		B-29	30,000	27.12	15.73	670	89.55
Single.....		....	60,000	38.35	22.24	670	89.55

single wheel curve and the B-29 dual wheel curves are identical to a depth of about 17 in. Also, for all practical purposes, the B-29 dual load and the 60,000-lb single load have become identical at a depth of about 80 in. Thus, the study of maximum vertical stresses establishes the two thicknesses at about 17 in. and 80 in. At depths of 17 in. and less, the tires of the 60,000-lb dual assembly induce maximum vertical stresses as independent 30,000-lb units. At depths of about 80 in. and greater, they induce maximum vertical stresses as a single 60,000-lb wheel load.

Fig. 31(b) shows the variation of maximum shearing stress with depth under a wheel load. For a given wheel load, the stress is greatest at the surface and decreases with depth. Also, at any given depth the greater wheel load induces the greater stress. Curves are shown for the maximum shearing stresses for 30,000-lb single wheels, 60,000-lb dual wheels, and 60,000-lb single wheels. It is noted that the curves for the B-29 dual wheel load and the 30,000-lb single wheel load coincide at about 20 in. It is believed that the two curves should be identical for depths less than 20 in., and the fact that the curves cross with the B-29 dual wheel showing less stress than the 30,000-lb single wheel in the range below 20 in. is attributed to an erroneous assumption of a weightless soil mass that was made in computing the stresses. It is also noted that at 70 in. the stress is approximately the same

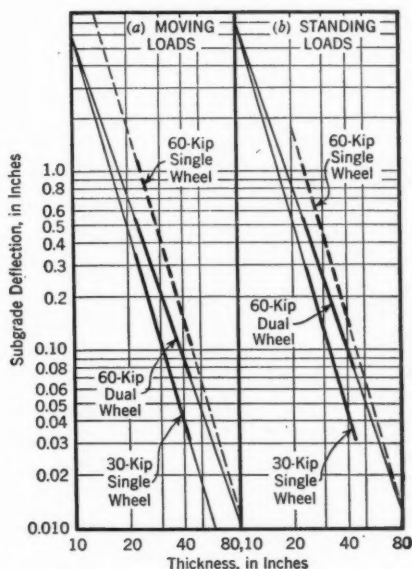


FIG. 35.—SUBGRADE DEFLECTION-THICKNESS CURVES; SINGLE WHEEL VERSUS DUAL WHEELS

for the B-29 dual wheels and the 60,000-lb single wheels. Thus, an analysis of maximum theoretical shearing stresses sets the limiting thicknesses at about 20 in. and 70 in. For depths of about 20 in. and less, the two tires on the B-29 dual wheel induce stresses as independent 30,000-lb wheel loads. For depths of about 70 in. and greater, the maximum shearing stresses induced by the B-29 dual wheel assembly are the same as those induced by a 60000-lb, single wheel load.

A comparison can also be made between single and dual wheels on the basis of subgrade deflections. In this paper the comparisons are made using a graphical method which is a special development. For this reason, it will be presented in more detail than were the comparisons which were based on vertical pressures and maximum shear stresses. Actual deflection data for a dual wheel load are available only from the Marietta test section. Although the Marietta data included comparisons between single wheels and dual wheels, none were obtained for a single wheel weighing 60,000 lb, equal to the combined maximum load on the B-29 dual wheel. Therefore, it was necessary to extrapolate the deflection data available to 60,000 lb for a single wheel to make the analysis. To justify such an extrapolation, deflection data will first be presented from the Stockton test section<sup>27</sup> in which data are available for single wheel loads between 5,000 and 53,500 lb.

Fig 32 is a plot of the subgrade deflections produced by standing wheel loads ranging from 5,000 lb to 53,500 lb in the Stockton tests.<sup>28</sup> (Open symbols represent measurements made prior to traffic and solid symbols represent tests made after 477 coverages with a 40,000-lb wheel load. Curves within the range of test data are shown by heavy lines, curves beyond the range of test data are shown by light lines; and interpolations and extrapolations are represented by dashed lines. Thickness refers to combined thickness of base and pavement.) In Fig. 32(a), subgrade deflections and pavement thicknesses are plotted as the coordinates for constant loads. In Fig. 32(b), the same data are plotted with subgrade deflection versus load for constant pavement thickness. Curves drawn approximately through the plotted points appear as straight lines on the logarithmic scales. By allowing slight deviations from the points, it is possible to draw the entire family of curves mutually parallel to each other so that any point selected on a curve in Fig. 32(a) will also lie on its corresponding curve in Fig. 32(b). Corresponding plots have been made for deflections measured under moving wheel loads but these are not presented in this paper. The curves are drawn with consideration of the entire group of points secured from all wheel loads, at all thicknesses of base, and for both moving and standing wheel load conditions. In some instances, individual curves do not pass through their plotted points as closely as might have been possible had the points been considered individually and not in connection with the entire group of deflection data.

The Stockton report demonstrates that the tire contact areas varied considerably for the different wheel loads; however, this variation apparently did not affect the deflections. In all cases, the depth of base was more than the

<sup>27</sup> "Report on Stockton Runway Test Section," U. S. Engr. Office, Sacramento, Calif., September, 1942.

<sup>28</sup> *Ibid.*, Figs. 9, 12, 16, 20, and 23.

width of the tire contact area and it is believed that under these conditions the size of the contact area has relatively little influence on the subgrade deflection. The same condition is noted in the Marietta deflection data.

By extending the curves for the pavement thickness shown in Fig. 32(b) from 53,500 lb to 60,000 lb as indicated, the theoretical deflections can be determined for each pavement thickness for the 60,000-lb wheel load. Replotting these points in Fig. 32(a), an extrapolated 60,000-lb wheel load can be drawn. The curve for the 30,000-lb load in Fig. 32(a) may be drawn by interpolating the deflections for a 30,000-lb load from the curves in Fig. 32(b).

The deflection data for moving wheel loads obtained at Stockton produce similar curves when plotted in this manner. The slopes of the curves for the moving wheel load deflections are the same as those for the standing wheel load deflections; however, the magnitudes of the deflections are less for the moving than for the standing wheel load.

The Stockton data show that the variation of subgrade deflection with thickness of base and pavement and the variation of subgrade deflection with wheel load can be expressed as straight lines on logarithmic plots for the range of wheel loads and thicknesses included in the study. With these straight-line curves interpolations are possible which are considered rational. Also, extrapolations are possible which are considered reasonable as long as they do not extend too far beyond the range of measured data.

Deflection measurements for single wheel and dual wheel loads from the Marietta data are shown in Figs. 33 and 34. (The plotted points are average values of all the standing wheel load tests—that is, with the loaded tire over the center of the gage—given in the report on the Marietta tests.<sup>39</sup> Fig. 33 shows deflections for a standing load and Fig. 34 those for a moving load. A 60,000-lb single wheel load curve obtained by extrapolation as discussed in preceding paragraphs is shown in Figs. 33 and 34. In the case of the B-29 dual wheel, the plots represent deflections secured with one tire, rather than with the center of a dual wheel, over a gage. For very thin pavements and base courses, this position of the tire over the gage would produce the greater subgrade deflection. At a greater thickness of pavement and base, the deflection under the center of the dual wheel is shown to be slightly greater than that directly under a tire. However, the difference is so small that it is within the range of the scattering of individual test data. Because the inclusion of deflection data for the case in which the center of the dual wheel is over the gage further complicates the analysis without appreciably changing the results, it has been omitted.

In Figs. 33(a) and 34(a), which are plots for single wheel loads, the curves drawn through the plotted points are mutually parallel. The slopes of the curves in Fig. 33(a) are the same as those in Fig. 34(a). They form parallel lines when plotted as indicated in Figs. 33(c) and 34(c). The plots for dual wheel loads, Fig. 33(b) and 34(b), are also all mutually parallel, but they are not parallel with the plots for single wheel loads, Figs. 33(a) and 34(a). Therefore, it is indicated, that, as the thickness of the base is increased, the deflections in

<sup>39</sup> "Certain Requirements for Flexible Pavement Design for B-29 Planes," U. S. Waterways Experiment Station, Vicksburg, Miss., Appendix on "Flexible Pavement Tests, Marietta, Ga.," 1 August 1945, Tables 3 and 10.

the subgrade will decrease in a different ratio for single wheels than for dual wheels.

In Figs. 33(c) and 34(c), which are deflection-versus-load plots for constant base and pavement thicknesses, the plotted curves for both single wheels and dual wheels are all mutually parallel. As the load on either a single or dual wheel is increased, the deflections produced by either will increase in the same ratio. The pavement, base, and subgrade therefore appear to be acting as an elastic medium in a comparable manner under either wheel.

Curves of deflection versus thickness for 30,000-lb and 60,000-lb single wheel loads and for 60,000-lb dual wheel loads are shown together in Fig. 35 for comparison. The curves for both the moving loads and the standing loads have been extended beyond the available data points to include thicknesses of base between 10 in. and about 80 in.

It is recognized that the extrapolation of the data to base thicknesses of 10 in. extends the curves to points where unreasonable deflections are indicated, because shear deformation would certainly have occurred if the subgrade had deflected the amounts shown by the extrapolated data. However, a basis of this extrapolation may be found in the Stockton test data, where deflections produced by wheel loads from 5,000 lb to 53,500 lb plot essentially as straight lines. If the test had been conducted on a subgrade with a sufficiently high CBR, it can be assumed that the deflections at a 10-in. depth of base would not have overstressed the subgrade. In this case the slope and the spacing of the curves would probably change, but it is believed that the change would be in the same proportion for the dual wheels as for the single wheels, with the result that the deflection-versus-thickness curves would still approach each other at about the same thicknesses as indicated by the available data. From Fig. 35, it can be noted that the curve for the B-29 dual wheel approaches the curve for a 30,000-lb single wheel at a thickness of base of about 75 in. Thus, for base courses 10 in. and less in thickness, a 60,000-lb, B-29, dual wheel tire produces subgrade deflections as an independent 30,000-lb single wheel; and, for bases thicker than 75 in., the 60,000-lb, B-29, dual wheel produces subgrade deflections as though it were a 60,000-lb single wheel.

On the basis of the deflection measurements and the theoretical analysis of stresses, the two tires of the B-29 dual wheel would act as a point load at a depth of about 75 in. Therefore, the assumption that, for all practical purposes, the 60,000-lb dual and single wheels produce stresses and strains in the subgrade at thicknesses of base and pavement greater than 75 in. is considered to be reasonably correct. In consequence, this assumption is used in the remainder of the analysis. The upper limit—the point where the effect of the B-29 dual wheel load (60,000 lb) is the same as that of a single 30,000-lb wheel load—is more difficult to determine. The deflection measurements indicate about 10 in.; the vertical pressures, about 17 in.; and the maximum shears, about 20 in. Since the 10-in. depth is slightly more conservative, it has been selected.

From these findings, it is concluded that the B-29 design curve should coincide with the 30,000-lb single wheel load curve at a depth of 10 in. and with the 60,000-lb single wheel load curve at a depth of 75 in. The thickness requirements between these two limits should vary in an orderly manner.

The next step is to develop the design curve from these limits. The single wheel load design curves presented in Symposium paper No. 9 are used for this development, rearranged with wheel load plotted against thickness to logarithmic scales as shown in Fig. 36. CBR values for single wheel loads are shown as a nest of curves.<sup>33</sup> To develop the B-29 curve, the point at a thickness of 10 in. with a wheel load of 30,000 lb is connected with the point at a 75-in. thickness with a 60,000-lb wheel load. A straight line is used to join these two points. Since the deflection data plot as straight lines on logarithmic scales, it was considered that a straight line between these points represented an orderly variation. The intersections of this straight line and the CBR curves determine the thickness requirements for the B-29 curve. The thickness required for a CBR of 3 is 41 in.; for a CBR of 4, 34 in.; etc. These thicknesses can be plotted to a semilogarithmic scale similar to the single wheel design curves presented in paper No. 9.

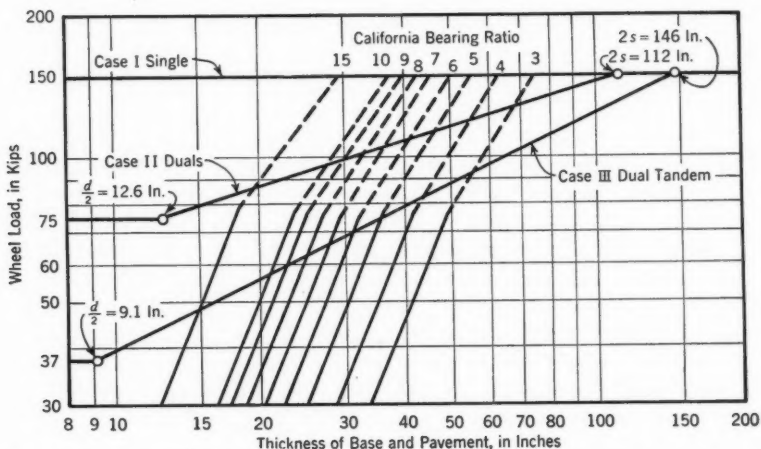


FIG. 36.—DEVELOPMENT OF DESIGN CURVES FOR A 60-KIP LOAD ON B-29 DUAL WHEELS

In translating thickness requirements from single to dual wheels in this manner, it is assumed that a dual wheel which produces the same maximum stresses and strains in the subgrade as a given single wheel will require the same thickness of base and pavement under actual traffic as does the single wheel. Theoretical considerations indicate that the requirements for the dual wheels will probably be slightly less than those for the single wheels.<sup>35</sup>

This method was extended to cover any multiple wheel assembly simply by resolving the two thicknesses—the shallow depth at which the dual wheels act as independent units and the deep depth at which they act as a single wheel—into ratios of appropriate dimensions of the assembly. In this manner the two depths can be determined for any assembly, and curves can be drawn.

From Fig. 30, it is readily apparent that the dimension of the assembly that governs the depth at which the dual wheel loads act as independent units is the spacing between the contact areas of the tires, which has been designated as  $d$ .

If the two tires are moved closer together, the stress patterns would overlap at a shallower depth. If they are moved farther apart, the depth would be greater before they overlap. Therefore, the depth at which the tires act as independent units has been resolved into a ratio of the clear distance between the contact areas of the two tires. This distance on the B-29 plane is approximately 20 in. and the thickness at which the tires act as independent units has been determined as 10 in. The ratio of this thickness in terms of  $d$  is 1 to 2 and can be expressed as  $d/2$ .

By inspection it can also be noted that the depth at which the dual wheels act as a single wheel load is governed by the distance between the centers of the wheels, which has been designated  $s$ . The tire contact width would also affect this depth, as wider tires would produce greater overlapping of stresses. However, at the depth under consideration, the percentage increase in overlapping of the stresses caused by wider tires would be small. Consequently, the depth at which the dual wheels act as a single unit is resolved into a ratio of the distance between the centers of the wheels. The spacing  $s$  between the centers of wheels on the B-29 dual wheels is 37.5 in. and the depth at which the dual wheel loads act as a single wheel load has been determined as 75 in. The ratio of this thickness to  $s$  is 2 to 1, and can be expressed as  $2s$ .

These ratios give the two depths for determining the end points of the design curves for any wheel assembly. At depths of  $d/2$  or less, the curve will be the same as that for a single wheel load equal to the individual wheel load in the assembly. At depths of  $2s$  and greater, the curve is the same as that for a single wheel load equal to the total weight on the assembly. In applying the ratios to dual tandems or other variations involving more than one dimension, the minimum distance  $d$  and the maximum distance  $s$  must be used. For a dual tandem assembly, distance  $d$  would be the clear spacing between the contact areas of either the front or the rear tires, and distance  $s$  would be the diagonal center-to-center spacing between a front tire and the opposite rear tire.

Extension of the existing single wheel design curves should be limited to multiple wheel assemblies which use tires with pressures comparable to those used in developing the single wheel curves. The existing single wheel curves are based on observations of the behavior of pavements subjected to traffic of tires inflated to pressures ranging from about 50 lb per sq in. to 110 lb per sq in. It is considered that the existing single wheel curves are applicable for tire pressures in the range of about 100 lb per sq in.

For a numerical illustration, consider the problem of a 300,000-lb (gross weight) plane. For all practical purposes, it is assumed that the total load will be carried on two wheel assemblies which are loaded to 150,000 lb each. Three different types of wheel assemblies are considered: Case I, a single tire; case II, dual wheels spaced 56 in. from the center to center; and case III, dual tandems with the wheels spaced 40 in. from center to center and the axles 61 in. from center to center. For each case it is assumed that the tires will have circular contact areas rather than the usual elliptically shaped areas. Studies have shown that this can be done without causing appreciable errors. A contact pressure of 100 lb per sq in. is used. With these data, values for  $d/2$  and  $2s$  can be computed readily. For case I no values are obtained, since this is the single

wheel. Values of  $d/2$  are 12.6 in. and 9.1 in. for cases II and III, respectively. Values of  $2s$  are 112 in. and 146 in. for cases II and III, respectively.

In Fig. 37, the single wheel design curves are shown with wheel load plotted against thickness of base and pavement to logarithmic scales. The curves have been extended to a 150,000-lb wheel load, using the tentative 150,000-lb curve introduced in the *Engineering Manual for War Department Construction* in July, 1946.<sup>33</sup> For case II the point at 75,000 lb and 12.6 in. ( $d/2$ ) was joined with the point at a 150,000-lb wheel load and 112 in. ( $2s$ ).

The intersections of this line and the CBR curves determine points on the design curve for this assembly. For a CBR of 3, 68 in. would be required; for a CBR of 4, 55 in.; etc. The design curve for case III was determined in a similar manner by joining the point at a 37,500-lb wheel load and 9.1 in. with the point at 150,000-lb wheel load and 146 in.

The intersections of this line and the CBR curves determine the thickness requirements for this assembly. For a CBR of 3, 56 in. is required; for a CBR of 4, 44 in.; etc. An interesting feature is that small variations can be made in the values of either  $d/2$  or  $2s$  without appreciably changing the thickness values for the design curves. Variations as great as 10% in either  $d/2$  or  $2s$  will not vary the thickness requirements more than about 1 in.

Comparisons of the effectiveness of the various assemblies can be made directly from this plot; however, it is difficult to obtain the actual extent of savings in thickness reduction unless they are converted to percentages. Table

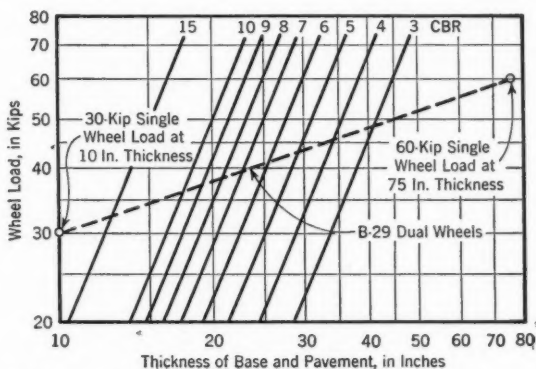


FIG. 37.—TENTATIVE METHOD OF COMPARING THICKNESS REQUIREMENTS FOR VARIOUS WHEEL LOAD ASSEMBLIES

TABLE 11.—COMPARISON OF THE EFFECTS OF ASSEMBLIES

Case	Wheel assembly	CBR = 4		CBR = 10	
		Thickness <sup>a</sup> (in.)	Decrease <sup>b</sup> (%)	Thickness <sup>a</sup> (in.)	Decrease <sup>b</sup> (%)
I	Single.....	63	..	38	..
II	Dual.....	55	13	28	26
III	Dual tandem.....	44	30	21	45

<sup>a</sup> Required thickness of base and pavement. <sup>b</sup> Percentage decrease from case I.

11 shows values illustrating the comparative effectiveness for two subgrades, one with a CBR of 4 and one with a CBR of 10. It shows the thickness of base and pavement required for each case and the savings as compared to the thickness required for case I, the single wheel. For a CBR of 4, a thickness of 63 in.

is required for the single wheel, whereas the dual wheels would require 55 in. and the dual tandem wheels only 44 in. These values represent decreases of 13% and 30% from the 63 in. required for the single wheel assembly. For a CBR of 10 the savings are even greater. The thickness required for the single wheel is 38 in., whereas 28 in. is required for the dual wheels and 21 in. for the dual tandem wheels—decreases of 26% and 45% respectively. For either CBR value, the dual tandem assemblies show considerably more percentage decrease than do the dual wheels (30% as compared to 13%, and 45% as compared to 26%). From the standpoint of the airfield pavement design, the dual tandem assemblies are more effective for this plane than are the dual wheels, because they result in a greater decrease in thickness requirements.

The preceding example shows that the method can be used to evaluate, readily, the effectiveness of various assemblies or the spacing of wheels in any given assembly. The design curves developed for the three cases should be considered merely as numerical examples that may not apply to any given plane. As noted previously, certain simplifying assumptions were necessary to develop the various curves. The assumptions are discussed in detail elsewhere.<sup>35</sup> The assumptions appear reasonable; however, each person using the method should evaluate their validity for himself. Although this method has these and other limitations, the basic reasoning is considered sound. The ease with which the method can be used, and the fact that minor variations in setting the two limiting thicknesses do not appreciably affect the design curves, justify its use for determining flexible pavement thickness requirements for multiple wheel assemblies until a more rational method is developed.

## APPRAISAL OF THE CBR METHOD

BY W. J. TURNBULL,<sup>40</sup> M. ASCE

The preceding papers have presented a review of the historical development of the CBR test and its application to flexible pavement design. They also contain individual summaries of the principal traffic test sections that were constructed to check the validity of the design curves. Finally, papers are included which verify the design curves for single wheel loads and which develop principles from which design curves for any desired multiple wheel load can be prepared. It is the purpose of this concluding paper to evaluate the several component parts of the design method and to present both its virtues and its shortcomings to the end that the maximum intelligent use may be made of the method in the laboratory, for design, and on construction. It is proposed to review the method as developed, thus pointing the way to its intelligent use.

The CBR test used in this method consists of forcing a 3-sq-in. circular piston into the soil and measuring the resistance to penetration. The resistance is converted to a bearing ratio by comparing it to the resistance obtained in penetrating a high-bearing material adopted as a standard. The penetration test is essentially a simple shear test, and the CBR is an index of the shearing strength of the soil. The CBR is used to determine, by design curves, the total thickness of base and pavement required to prevent shear deformation in the given subgrade soil for a specified wheel load.

After the initial trial of the CBR method by the Corps of Engineers, it became apparent that conditions and soil types, varying considerably from those encountered in California, made it necessary to initiate a program of laboratory research on the test used in the method. This project, explained in paper No. 4, was assigned to the Soils Division of the Waterways Experiment Station. After about 3 years of study, the test was developed to the state in which it is now (1948) used by the Corps of Engineers. Also, as presented in several previous papers, a comprehensive program of field testing was conducted to correlate the design curves with traffic requirements. Research and field correlation studies have been carried to the point where it is believed that the CBR method can be used for the practical and economical design of military airports.

The studies of the California method of design have resulted in considering the method to have three separate parts:

1. The design curves,
2. The penetration test, and
3. The preparation of laboratory samples.

## THE DESIGN CURVES

The original design curves for airplane loadings were extrapolations of the basic highway loading curves developed by the California State Highway

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Department. These curves have been correlated with field behavior by measuring the CBR, in place, of test sections and existing pavements and by observing the effect of traffic. The curves have been adjusted to the point where it is considered that, if in-place CBR values for a specified base and subgrade are available, the curves will reflect their behavior under traffic. In other words, when the field in-place test indicates underdesign, overdesign, or correct design, by the present curves, the actual results of pavement behavior under traffic have generally substantiated the curves. This is particularly true for fine-grained cohesive soils. Some scattering of data has been experienced for cohesionless materials, especially for clean sands, and the design curves are generally considered conservative for these materials. As shown in paper No. 9, the curves, as drawn, conform closely to the available data and produce an orderly group of curves. Extreme accuracy should not be read into the curves; generally, they could be shifted an amount equivalent to 1 in. or 2 in. of pavement thickness and would still agree reasonably well with the data. In their present state, the curves are subject to minor modification from time to time as additional data become available. The final report on Stockton test section No. 2<sup>41</sup> in which loads up to 200,000 lb were tested (traffic tests completed in 1947) has not as yet (1948) been published, however the data indicate that curves extrapolated from the lower wheel loads (up to 70,000 lb) for the high loads used in the Stockton tests, were not conservative.

One criticism of the California method is that the base thicknesses obtained by this method are too great. As previously stated, the thicknesses indicated by the design curves are not excessively conservative. Also, it is noted that no safety factor of additional thickness has been included in the design curves. The criticism is partly the result of a lack in understanding the basic design premise with respect to permanent construction as used by the Corps of Engineers. Permanent construction may be defined as resulting in stable pavements for continuous use by the designated wheel load for a long period with only minor maintenance. Permanent construction is obtained by using a conservative design which (a) assumes maximum field moisture in the base and the subgrade, and (b) requires high-quality upper base and wearing courses. In general, permanent construction as defined is considerably more conservative than either highway or airport design in the years preceding World War II. This is not a criticism of flexible pavements designed by agencies other than the Corps of Engineers but does indicate a probable difference in the definition or interpretation of anticipated life. It is realized that many arguments can be advanced against a pavement design meeting the requirements for permanent construction as defined by the Corps of Engineers; however, it has been, and still is, considered feasible to design permanent military airports in the continental limits of the United States by this criterion. By clarifying the basic premise of design, it is hoped that engineers who have believed the CBR method

<sup>41</sup> "Flexible Pavement Test Section for 300,000-lb. Airplanes, Stockton, California," by Ralph A. Freeman and O. J. Porter, *Proceedings, Highway Research Board, National Research Council*, Vol. 25, 1925, p. 23.

to result in unduly costly and conservative pavements may reconsider the test in a different light.

Another feature of the CBR method of design which is sometimes criticized is that a base composed, throughout, of a high-quality material is not considered to have any advantage over the same thickness of layered base with inferior materials in the lower part. The present studies have progressed far enough to give good indications that base materials do not all have the same stress-distributing qualities and that low-quality bases are probably poorer stress-distributing media than are high-quality bases. It is entirely possible that the thickness indicated by the design curves should be increased if a low-quality base is used in the lower part of the pavement sections.

#### THE PENETRATION TEST

Part 2 of the method, as previously mentioned, deals with the penetration test. This test is relatively simple with very little chance for the personal element to affect the results. Experimentation on the mechanics and the procedures in making the penetration test, both in the laboratory and in the field, has indicated that it is reliable and that results can be reproduced reasonably well in the hands of ordinarily good technicians. Special details on technique are summarized in paper No. 4.<sup>4</sup> In testing clean sands, it is necessary to surcharge the surrounding area during the test to obtain CBR values that are indicative of the behavior of the sands. Difficulties are experienced in reproducing results on materials such as clay-gravels which contain large-size particles—because large particles directly under the piston affect the results. A large number of tests on these materials and an average of the results will assist materially in obtaining good test values. Soil is always a variable material and few uniform deposits exist. In using the CBR test, it is necessary, as with any other test, to conduct a sufficient number of tests to obtain good average results.

Admittedly, the penetration test has the weaknesses of all empirical-type tests. However, it has the following advantages:

- a. It is supported by a background of practical experience;
- b. It is applicable to a wide range of materials;
- c. The equipment used in the test is light and mobile; and
- d. The test can be used in the laboratory and field for correlation, design, evaluation, and construction control purposes.

Although they are not a part of the penetration test as such, mold effects have a direct bearing on the CBR values obtained. Mold effects have been harassing and are continuing to be studied. These effects are more noticeable in silts than in clays. It is necessary to exercise good judgment in weighing possible mold effects on CBR values for use in design.

#### PREPARATION OF LABORATORY SAMPLES

Part 3 of the method—that is, the preparation of the samples for the penetration test—is the most troublesome in the use of the CBR method as a design

tool. The problem consists in preparing the sample so that it truly represents the condition of (1) consolidation, (2) moisture content, and (3) the structure that will be obtained in the prototype. If a true representation is obtained of these three conditions, then it has been proved that the CBR design curves can be used with assurance. It is admitted that an entirely satisfactory laboratory method of preparing specimens to duplicate the conditions in the prototype has not been developed. When tempered with judgment, however, the method can be used to give samples that will show CBR values which approximate reasonably well the CBR that can be expected in the prototype. As with the penetration test, greater trouble is experienced with sands and granular materials than with fine-grained cohesive soils.

The method of preparing specimens to the condition or consolidation expected in the prototype has consisted of compacting the specimens with a standardized compactive effort. It was desired to set the compactive effort high to produce specimens with a high degree of consolidation (which is beneficial if obtained in the prototype because the compaction increases the CBR of the material), thus permitting thinner base courses. This high compaction also reduces the possible consolidation hazards under the heavier wheel loads. Therefore, the Corps of Engineers increased the standard AASHTO compactive effort to such a degree that the resultant maximum density was believed to be sufficiently high to meet the requirements necessary for satisfactory field behavior. The revised test is commonly called the modified AASHTO compaction test. If the design CBR is based on a highly compacted sample and this compaction is not obtained in the prototype, the actual CBR obtained in construction will be less than the design value and the result will be a pavement section that is underdesigned. It must be admitted that the compactive effort used in the original CBR test as developed by the California Division of Highways and the compactive effort adopted by the Corps of Engineers produce densities in laboratory specimens that are very difficult to obtain with the usual construction equipment and methods. This fact is the source of some criticism.

In this connection it should be recognized that in the California method the emphasis has been placed on design thickness. This is not intentional, but results from the fact that a special test and a set of design curves are used for determining the required thickness. However, it is just as important that the subgrade and the base be compacted adequately to prevent detrimental consolidation. The CBR test does not provide information with respect to the amount of settlement to be expected in the pavement surface as a result of consolidation in the base course and in the subgrade. It is quite possible to design and to construct a pavement section containing a material compacted to a degree which may be satisfactory from the standpoint of shear deformation but which may allow consolidation to take place to the point that detrimental settlement will be produced at the surface. For this reason, it is necessary to supplement the CBR test with definite compaction requirements. The CBR method as used by the Corps of Engineers requires compaction to definite percentages of the density obtained from laboratory or field compaction tests. The percentages are varied in accordance with the wheel load and the depth

below the surface to the given soil. It is recognized that for some soils the specified percentages cannot be obtained practicably. In these cases it is necessary to adjust the design to obtain a balance between the thickness requirements and the compaction requirements. Method 2, for preparing and testing CBR specimens, as described in paper No. 4, will furnish information that will permit ready adjustments of these requirements.

One of the favorable features of the CBR test is that it can be conducted on a laboratory specimen in which the moisture content can be adjusted to a given design value—condition (2) in the problem of preparing samples to meet prototype conditions. This phase is simplified, in that, for design the condition of maximum field moisture has been assumed to exist. Maximum field moisture is the maximum moisture content that a given soil can reach under conditions of heavy rainfall, high water table, and poor drainage. In general, under the condition of maximum field moisture, the voids in the soil, except for sands, are only from about 75% to 95% (by volume) filled with water. Consequently, the design condition is not as conservative as if it were based on a completely saturated condition where all the voids were filled with water. As information on moisture conditions under pavements was very conflicting, the Corps of Engineers had no other course than to assume the maximum condition to insure conservative design.

To simulate the maximum field moisture, specimens are soaked in the laboratory in water for 4 days prior to the penetration test. In general, 4 days of soaking approaches the maximum field moisture, especially near the surface where the penetration tests are made. In the prototype the condition of maximum field moisture may occur: (a) Immediately; (b) over a period of years; or (c) never, in some instances where ground water, drainage conditions, and soil types are particularly favorable. In conditions (b) and (c), the CBR method tends to be criticized because its use results in pavements which are too conservatively designed. The basis for this criticism in the case of condition (b) is not believed to be valid in that surplus strength is present in the pavement only until the design moisture condition occurs. However, in the case of condition (c), the criticism is believed to be valid. Long-range studies are being conducted to determine the moisture content that occurs under pavements in arid, semi-arid, and wet regions.

It is true that, in general, the condition of maximum field moisture represents the lowest strength of a soil; however, when compacted to the degree called for by the design, the resulting strength may usually be relied on to fulfil the design requirements. Only in the cases of highly expansive soils, frost effects, or soils compacted to a much lower degree of density than called for by the design, are wholly inadequate strengths obtained.

At this point it is believed desirable to demonstrate the possible effects of consolidation on the CBR of a given subgrade soil or a base layer when a high percentage of the voids is filled with water. When consolidation occurs under this condition, hydrostatic excess pressures can be developed which may very materially reduce the strength of the soil. This reduction of strength can occur quite rapidly. The possibility of developing hydrostatic excess pressures

is greatest for silts and fine silty sands, which are relatively impermeable but which, as they possess very little cohesion, have a tendency to consolidate readily under load. High water table, capillarity, pumping, vapor condensation, and poor drainage are critical factors in the development of hydrostatic excess pressures.

Under condition (3) in preparing samples to meet prototype conditions, the problem is to produce the same structure in laboratory samples as that produced by field compaction. As in most research and test programs, the research on the CBR test has demonstrated the need for additional lines of study—one of the most important being the possible variation in structure that a soil may attain as the result of variation in degree and type of compactive effort and of variation in compactive moisture. Variation in structure may indicate a considerable change in strength. Change in strength due to change in initial molding moisture has been demonstrated by the laboratory study on the CBR test, details of which may be obtained from a paper entitled, "Stress-Strain Characteristics of Compacted Soil Systems," by Joseph B. Eustis, Assoc. M. ASCE, and John L. McRae, presented at the ASCE Annual Meeting in New York, N. Y. on January 19, 1945. The Waterways Experiment Station engaged in a series of field compaction studies in an attempt to determine, among other things, any possible variation in structure produced by various types of field compaction equipment in comparison with that produced by various types of laboratory compaction equipment and methods. Soils studied were sand, sand-clay, and clay-silt. Structure effects were evident in the field studies on the two finer-grained materials. On one of them, reasonable correlation was obtained with the dynamic laboratory test, and on the other it was not. One very noticeable effect of the field "density-versus-moisture" curves as compared to those of the laboratory is that the former were transposed to the right, closer to the zero air-voids curve. Until this condition is corrected, complete correlation between field and laboratory compacted structures cannot exist.

#### DISCUSSION

In applying the CBR method of design, it is necessary to make a thorough study of the existing subgrade soils and of the possible materials available for base course construction. The materials to be used must first be selected and then the proper CBR value determined for each. It is obvious that the selected CBR value must be based on the density and moisture conditions to be expected in the prototype if a satisfactory pavement is to be obtained. A high degree of field compaction is desirable, in that subsequent consolidation due to plane traffic is held to a minimum and higher CBR values are obtained.

In the selection of the proper CBR value for design, the Corps of Engineers resorts to one or the other of two test procedures. The first might be called the minimum procedure and is generally used, particularly in emergency construction, where time for design is limited. The second or longer procedure is a

more complete development of design CBR values and is desirable if the time element is not too critical, and particularly where it is advantageous to have a complete picture of the variation of the CBR with both density and water content. Both of these methods are demonstrated in paper No. 4.

In the minimum (method 1) CBR test procedure, the compaction curve is fully developed for the modified effort and densities are obtained for two lesser compactive efforts at modified optimum moisture. CBR values on three soaked specimens molded at optimum water content for the three compactive efforts are obtained. Thus, at optimum moisture the range in CBR values with density is obtained. The variation of the CBR with moisture is not obtained; consequently, unless very close control of the moisture in field construction is maintained, considerable variation in strength would result.

The longer CBR test procedure (method 2) consists in developing compaction curves for the modified compactive effort, and for two lesser efforts, and determining the CBR of each compacted specimen. This procedure is demonstrated by Fig. 11. The data plotted in Fig. 11 will furnish design engineers and field engineers with a visual picture of the possible spread of CBR test values with both the water content and the density of the soil. The design engineer thus has a better method for choosing the design CBR value, and the field engineer has a better tool for the control of construction. By building up a background of experience and familiarity with the soils in a given area, the design engineer can evaluate the results of the CBR tests and can select a value that he can expect to obtain in the prototype—one which he can use with confidence. One important feature that should be emphasized is that tests on all soils except those for clean, granular materials tend to show a rapid decrease in soaked or unsoaked CBR values with increasing molding moisture above optimum. One must be impressed by the implication that field compaction and moisture control can be very critical in determining the final strength of the subgrade and the base course.

Briefly, it may be stated that, as a result of the laboratory research program, field traffic tests, and prototype experience behavior, the Corps of Engineers has found the CBR test to have broad utility, in that it can be used successfully as a laboratory design test, a field construction control test, and a pavement evaluation test.

*Summary.*—The penetration test and the design curves have been developed to the point where they are considered very reliable. It must be admitted that an entirely satisfactory method has not been developed for preparing laboratory specimens that will duplicate prototype conditions for all soils at all times, under all conditions, and for all areas. In particular the Corps is making a serious effort to determine in what general areas and specific localities the conservatism of design as represented by the 4-day soaking period need not be followed. However, with due consideration of the limitations just mentioned, considerable progress has been made in the preparation of specimens, and the test procedures and methods of data analysis now proposed give the design engineer very good information on the CBR that he can expect to obtain in the

prototype. It is not considered amiss to point out that the difficulties encountered in preparing specimens for the CBR test would also be encountered in any other type of test.

In conclusion, it is desired to state that the Corps of Engineers believes that the empirical California method, as presently developed and when properly applied, is reasonable and satisfactory for the design of flexible military airport pavements. This statement does not imply that a more rational method of design employing actual strength characteristics of the soil is not desirable. It is a matter of record that the Corps has several long-range research and fact-finding studies under way from which it is hoped that a completely rational design procedure may be developed.

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# AMERICAN SOCIETY OF CIVIL ENGINEERS

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## PAPERS

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### INTEGRATING THE EQUATION OF NONUNIFORM FLOW

BY M. E. VON SEGGERN,<sup>1</sup> ASSOC. M. ASCE

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#### SYNOPSIS

A complete integration of the equation of nonuniform flow would appear to be highly desirable, as an integration method makes it possible to analyze quickly problems that the average engineer avoids because of the mass of tedious computations that would be required by other methods. This paper presents a complete and simple integration method adapted to all types of cross sections ordinarily encountered in practice. The hydraulic exponent,  $n$ , which was introduced by Boris A. Bakhmeteff,<sup>2</sup> Hon. M. ASCE, is used in this presentation exactly as it is used by him. In addition, however, a secondary exponent,  $m$ , is employed to set up the equation of nonuniform flow in such form that it can be completely evaluated by integration. Also presented is a novel and time saving method of computing the exponents,  $n$  and  $m$ .

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#### INTRODUCTION

Although various attempts have been made to solve, by integration, hydraulic problems involving nonuniform flow, most of the early efforts have only an historical interest and will not be mentioned. Perhaps the best known of these was that of J. A. C. Bresse,<sup>3</sup> in which the Chézy friction formula is used and the channel is idealized into a rectangular channel of infinite width. More recently, Professor Bakhmeteff<sup>2</sup> devised an integration method that accommodates all shapes of channels and allows the user to select his own flow formula. The Bakhmeteff method, however, evaluates only a part of the integral, thus making it necessary to apply a factor or a series of factors to account for the remainder. Nagaho Mononobe<sup>4</sup> presented an integration method that com-

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NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **June 1, 1949**.

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<sup>2</sup> "Hydraulics of Open Channels," by Boris A. Bakhmeteff, McGraw-Hill Book Co., Inc., New York, N. Y., 1932.

<sup>3</sup> "Hydraulics of Steady Flow in Open Channels," by S. M. Woodward and C. J. Posey, John Wiley & Sons, Inc., New York, N. Y., 1941, pp. 63-74.

<sup>4</sup> "Back-Water and Drop-Down Curves for Uniform Channels," by Nagaho Mononobe, *Transactions, ASCE*, Vol. 103, 1938, p. 950.

pletely evaluates the integral and includes all channel shapes, but this method contains approximations which are deemed inadmissible. A complete integration of the differential equation of nonuniform flow has been developed by S. M. Woodward, Hon. M. ASCE, and C. J. Posey,<sup>5</sup> M. ASCE, for channels with horizontal bottom grade, but this integration is not applied to channels with sustaining slopes.

The method presented herein is, however, capable of such extensions through use of the Bakhmeteff exponent  $n$  and of a secondary exponent  $m$  to set up the equation of nonuniform flow in such form that it can be completely evaluated by integration. Inasmuch as it is presumed that the reader is familiar with the general features of open channel flow, neither the types of surface curves nor the conditions under which they are formed are discussed.

*Notation.*—The letter symbols used in this paper are defined where they first appear, in the text or in illustrations, and are assembled for convenience of reference in Appendix I.

#### EQUATION OF NONUNIFORM FLOW

"Nonuniform" flow, sometimes referred to as "varied" flow, is defined briefly as steady flow in a channel where the water surface and the energy gradient are not parallel to the bottom of the channel. All types of backwater curves come under this heading, which is distinguished from "uniform" flow, where the water surface and the energy gradient are parallel to the bottom. Inasmuch as every standard textbook dealing with open channels develops the equation of nonuniform flow, that derivation will not be repeated here.

The equation of nonuniform flow can be written as

$$S_s = \frac{V^2}{C^2 R} + \frac{d}{dx} \left( \frac{V^2}{2g} \right) \dots \dots \dots (1a)$$

or

$$\frac{dy}{dx} = S_0 \frac{1 - (K_0/K)^2}{1 - \frac{Q^2 b_w}{g A^3}} \dots \dots \dots (1b)$$

or

$$dx = \left( \frac{1 - \frac{Q^2 b_w}{g A^3}}{S_0 - S_f} \right) dy \dots \dots \dots (1c)$$

—each of which is an identical expression if

$$K = A C \sqrt{R} \dots \dots \dots (2a)$$

and

$$S_s = S_0 - \frac{dy}{dx} \dots \dots \dots (2b)$$

In Eqs. 1 and 2,  $S_s$  is the slope of the water surface;  $V$  is the velocity of flow;  $C$  is the Chézy coefficient of flow;  $R$  is the hydraulic radius of the cross-sectional area;  $g$  is the acceleration due to gravity;  $x$  is the distance along the channel

<sup>5</sup>"Hydraulics of Steady Flow in Open Channels," by S. M. Woodward and C. J. Posey, John Wiley & Sons, Inc., New York, N. Y., 1941.

parallel to the bottom;  $y$  is the depth of flow;  $K$  is the conveyance of a cross section;  $Q$  is the discharge or rate of flow;  $b_w$  is the width of the water surface;  $A$  is the cross-sectional area;  $S_f$  is the slope of the energy grade line; and the subscript 0 denotes the value of a term for uniform flow conditions. For any specific value of  $Q$ ,  $K_0$  is constant and  $S_0$ , being the slope corresponding to uniform flow, equals the bottom slope of the channel.

Borrowing from the notations used by Professor Bakhmeteff, in which

$$M = \sqrt{A^3/b_w} \dots \dots \dots (3a)$$

and<sup>6</sup>

$$M_c = (\sqrt{A^3/b_w})_c = \sqrt{Q^2/g} \dots \dots \dots (3b)$$

—in which  $M$  is a function of the cross section and the subscript  $c$  denotes the value of a quantity at critical depth—Eq. 1b reduces to

$$\frac{dy}{dx} = S_0 \frac{1 - (K_0/K)^2}{1 - (M_c/M)^2} \dots \dots \dots (4)$$

Into Eq. 4 Professor Bakhmeteff introduces the relation:

$$\left(\frac{K_0}{K}\right)^2 = \left(\frac{y_0}{y}\right)^n \dots \dots \dots (5a)$$

which as he demonstrates holds remarkably well for most channels, and in which  $y_0$  is the depth for uniform flow and  $n$  is a hydraulic exponent.<sup>7</sup> The writer now proposes the relation—

$$\left(\frac{M_c}{M}\right)^2 = \left(\frac{y_c}{y}\right)^m \dots \dots \dots (5b)$$

—which is about as accurate as Eq. 5a, and in which  $y_c$  is the critical depth and  $m$  is an exponent similar to  $n$ . A discussion of the values of the hydraulic exponents  $m$  and  $n$  is presented elsewhere in this paper. It will be noted that these two exponents are the same as the exponents  $a$  and  $b$  defined by Mr. Woodward and Professor Posey.<sup>8</sup>

With these two additional substitutions (Eqs. 5), Eq. 4 becomes

$$\frac{dy}{dx} = S_0 \frac{1 - (y_0/y)^n}{1 - (y_c/y)^m} \dots \dots \dots (6)$$

By designating

$$u = y_0/y \dots \dots \dots (7a)$$

and

$$v = y_c/y \dots \dots \dots (7b)$$

so that

$$y = y_0/u = y_c/v \dots \dots \dots (7c)$$

<sup>6</sup>"Hydraulics of Open Channels," by Boris A. Bakhmeteff, McGraw-Hill Book Co., Inc., New York, N. Y., 1932, p. 35.

<sup>7</sup>*Ibid.*, p. 84.

<sup>8</sup>"Hydraulics of Steady Flow in Open Channels," by S. M. Woodward and C. J. Posey, John Wiley & Sons, Inc., New York, N. Y., 1941, p. 80.

it is found that

$$v = u (y_c/y_0) \dots\dots\dots (7d)$$

and

$$dy = - (y_0/u^2) du \dots\dots\dots (7e)$$

Substituting Eqs. 7d and 7e, Eq. 6 gives

$$dx = \frac{1}{S_0} \frac{1 - \left(\frac{y_c}{y_0} u\right)^m}{1 - u^n} \left(-\frac{y_0}{u^2}\right) du \dots\dots\dots (8a)$$

which can be reduced to

$$dx = \frac{y_0}{S_0} \left[ \frac{(y_c/y_0)^m u^m}{u^2 (1 - u^n)} - \frac{1}{u^2 (1 - u^n)} \right] du \dots\dots\dots (8b)$$

Integration of Eq. 8b gives

$$x = \frac{y_0}{S_0} \left[ \left(\frac{y_c}{y_0}\right)^m \int \frac{u^m du}{u^2 (1 - u^n)} - \int \frac{du}{u^2 (1 - u^n)} \right] \dots\dots\dots (8c)$$

and by designating

$$Y = \int \frac{u^m du}{u^2 (1 - u^n)} \dots\dots\dots (9a)$$

and

$$Z = - \int \frac{du}{u^2 (1 - u^n)} \dots\dots\dots (9b)$$

Eq. 8c becomes

$$x = \frac{y_0}{S_0} \left[ \left(\frac{y_c}{y_0}\right)^m Y + Z \right] \dots\dots\dots (10)$$

The integrals represented by  $Y$  and  $Z$  are both the sums of infinite series, therefore it is not practical to perform the integration for each particular problem. However, since the values of  $m$  for most cross sections lie between 3.0 and 5.0 and most of the values of  $n$  are between 2.0 and 5.4, tables can be computed giving the values of  $Y$  and  $Z$  for various values of  $m$ ,  $n$ , and  $u$  as explained in Appendix II. For a given problem,  $m$ ,  $n$ , and  $u$  are easily computed so that with the aid of such tables as Tables 1 and 2<sup>8a</sup> the solution is readily obtained.

By integrating between the limits  $u_1$  and  $u_2$  the distance between these points is obtained. Thus,

$$x_2 - x_1 = \frac{y_0}{S_0} \left[ \left(\frac{y_c}{y_0}\right)^m (Y_2 - Y_1) + (Z_2 - Z_1) \right] \dots\dots\dots (11)$$

and, by substituting,

$$\Pi = (y_c/y_0)^m Y + Z \dots\dots\dots (12)$$

<sup>8a</sup> Tables 1 and 2 are greatly condensed forms of the complete tables, confined to the values of  $m$ ,  $n$ , and  $u$  needed for solving the illustrative examples in the paper. The complete tables contain values of  $Y$  and  $Z$  for the ordinarily usable values of  $m$  from 3.0 to 5.0, values of  $n$  from 2.2 to 5.4, and values of  $u$  from 0 to 30. Copies of these tables are available on request at a price of \$1.00.

TABLE 1.—VALUES OF THE  
Y-FUNCTION

u	n			
	3.0	3.2	3.4	3.6
m = 3.0				
0.78	0.389	0.379	0.371	0.364
0.79	0.404	0.394	0.385	0.378
0.999	2.183	2.069	1.968	1.878
1.001	2.788	2.475	2.231	2.032
1.40	0.798	0.622	0.497	0.406
1.42	0.782	0.608	0.485	0.394
4.0	0.251	0.158	0.103	0.068
4.5	0.223	0.137	0.087	0.056
5.0	0.200	0.121	0.075	0.048
7.0	0.143	0.081	0.047	0.028
8.0	0.125	0.069	0.039	0.022
m = 3.2				
0.38	0.055	0.055	0.055	0.055
0.40	0.062	0.062	0.062	0.061
0.64	0.193	0.190	0.187	0.185
m = 3.4				
0.32	.....	0.027	0.027	0.027
0.34	.....	0.032	0.032	0.032
0.48	.....	0.075	0.074	0.074
0.50	.....	0.083	0.082	0.082
0.72	.....	0.226	0.222	0.218
0.73	.....	0.236	0.232	0.228
0.84	.....	0.385	0.375	0.366
0.85	.....	0.404	0.393	0.383
0.89	.....	0.497	0.481	0.467
0.90	.....	0.525	0.508	0.493
0.92	.....	0.593	0.572	0.554
0.93	.....	0.633	0.611	0.591
0.97	.....	0.894	0.857	0.825
0.975	.....	0.951	0.911	0.875
0.999	.....	1.954	1.856	1.769
1.001	.....	2.936	2.554	2.283
1.02	.....	1.998	1.671	1.450
1.03	.....	1.870	1.552	1.337
1.26	.....	1.178	0.907	0.725
1.28	.....	1.154	0.885	0.705
2.1	.....	0.704	0.485	0.348
2.2	.....	0.676	0.462	0.328
2.6	.....	0.588	0.388	0.267
2.7	.....	0.570	0.373	0.255

TABLE 2.—VALUES OF THE  
Z-FUNCTION

u	n			
	3.0	3.2	3.4	3.6
0.32	3.073	3.088	3.098	3.105
0.34	2.882	2.898	2.910	2.918
0.36	2.712	2.729	2.741	2.750
0.38	2.558	2.576	2.590	2.600
0.40	2.418	2.438	2.453	2.464
0.48	1.963	1.989	2.009	2.024
0.50	1.868	1.896	1.918	1.934
0.64	1.332	1.373	1.405	1.430
0.72	1.078	1.127	1.167	1.199
0.73	1.048	1.098	1.138	1.171
0.78	0.893	0.950	0.995	1.033
0.79	0.862	0.920	0.966	1.005
0.84	0.697	0.762	0.816	0.860
0.85	0.662	0.729	0.784	0.829
0.89	0.506	0.581	0.643	0.695
0.90	0.462	0.539	0.603	0.657
0.92	0.364	0.446	0.514	0.572
0.93	0.308	0.393	0.464	0.525
0.97	-0.019	0.086	0.173	0.249
0.975	-0.085	0.023	0.115	0.193
0.999	-1.182	-1.006	-0.855	-0.724
1.001	-1.932	-1.789	-1.664	-1.555
1.02	-0.882	-0.809	-0.746	-0.691
1.03	-0.747	-0.683	-0.629	-0.581
1.26	-0.144	-0.128	-0.114	-0.102
1.28	-0.132	-0.117	-0.104	-0.093
1.40	-0.096	-0.083	-0.073	-0.064
1.42	-0.090	-0.078	-0.068	-0.059
2.1	-0.017	-0.014	-0.011	-0.009
2.2	-0.014	-0.011	-0.009	-0.007
2.6	-0.007	-0.006	-0.004	-0.003
2.7	-0.006	-0.005	-0.004	-0.003
4.0	-0.001	-0.001	-0.001	-0.001
4.5	-0.001	-0.001	-0.000	-0.000
5.0	-0.001	.....	.....	.....
6.0	-0.000	.....	.....	.....
7.0	.....	.....	.....	.....
8.0	.....	.....	.....	.....

Eq. 11 becomes

$$L = x_2 - x_1 = \frac{y_0}{S_0} (\Pi_2 - \Pi_1) \dots \dots \dots (13)$$

in which  $L$  is the length of the surface curve. By definition,  $L$  is measured parallel to the bottom of the channel. For a channel with mild slope—that is, a slope less than critical—the distance measured in this manner differs but little from the horizontal distance. It is, therefore, convenient and customary to consider  $L$  as a horizontal distance in this case. For a channel with a steep slope, however, it is possible that there may be an appreciable difference between the slope distance and the horizontal distance.

In the form of Eq. 13, the equation of nonuniform flow will be found most convenient.

#### THE HYDRAULIC EXPONENTS $m$ AND $n$

Inasmuch as the hydraulic exponent  $n$  (see Eq. 5a) has been thoroughly explained elsewhere,<sup>7</sup> it is needless to discuss it in detail. The exponent  $m$  proposed herein is very similar to  $n$  (see Eq. 5b) so that for any two values of  $y$  and the corresponding values of  $K$  and  $M$  the values of  $n$  and  $m$  can be solved for directly by the relationships:

$$n = 2 \frac{\log K_1/K_2}{\log y_1/y_2} \dots \dots \dots (14a)$$

and

$$m = 2 \frac{\log M_1/M_2}{\log y_1/y_2} \dots \dots \dots (14b)$$

For a rectangular channel,

$$\left(\frac{y_1}{y_2}\right)^m = \left(\frac{M_1}{M_2}\right)^2 = \left(\sqrt{\frac{A^3_1/(b_w)_1}{A^3_2/(b_w)_2}}\right)^2 = \frac{A^3_1 (b_w)_2}{A^3_2 (b_w)_1} = \frac{(b y_1)^3 (b_w)_2}{(b y_2)^3 (b_w)_1} \dots (15)$$

However,

$$(b_w)_1 = (b_w)_2 = b \dots \dots \dots (16)$$

so that

$$(y_1/y_2)^m = (y_1/y_2)^3 \dots \dots \dots (17)$$

and  $m = 3$ . It can be shown similarly that for a V-shaped channel  $m = 5$ , and for a parabolic channel  $m = 4$ .

In most cases, however, it will be unnecessary to determine these exponents more exactly than to the nearest tenth. For this purpose it is convenient to plot  $\log K$  against  $\log y$ , and  $\log M$  against  $\log y$  for the channel in question. The slopes of the lines thus obtained will be one half of the values of the exponents.

In the case of  $m$ , the relation is based entirely on the shape of the channel. For instance, two channels with the same side slopes, but having different bottom widths, will have the same value of  $m$  for values of  $y$  between 5 and 8 in the case of a bottom width of 10 and for values of  $y$  between 10 and 16 in the case of a bottom width of 20. Using this principle it is possible to plot  $\log M$  against  $\log y$  for a channel of unit bottom width, and from the plotting to determine the values of  $m$  for any range of depths or bottom widths. Fig. 1 is a graph of this type for channels of unit bottom width with different side slopes.

To illustrate the use of Fig. 1 in determining  $m$ , assume that it is desired to determine  $m$  for the range of depths from  $y_1 = 10$  to  $y_2 = 15$  in a channel having a bottom width of 10 and side slopes of 1 on  $1\frac{1}{2}$ . First divide the depths  $y_1$  and  $y_2$  by the bottom width, giving 1.0 and 1.5, respectively. In Fig. 1, lay a triangle across the points where the 1 on  $1\frac{1}{2}$  slope line crosses the 1.0 value and the 1.5 value of  $y/b$  (see insert in Fig. 1). With the aid of a straight

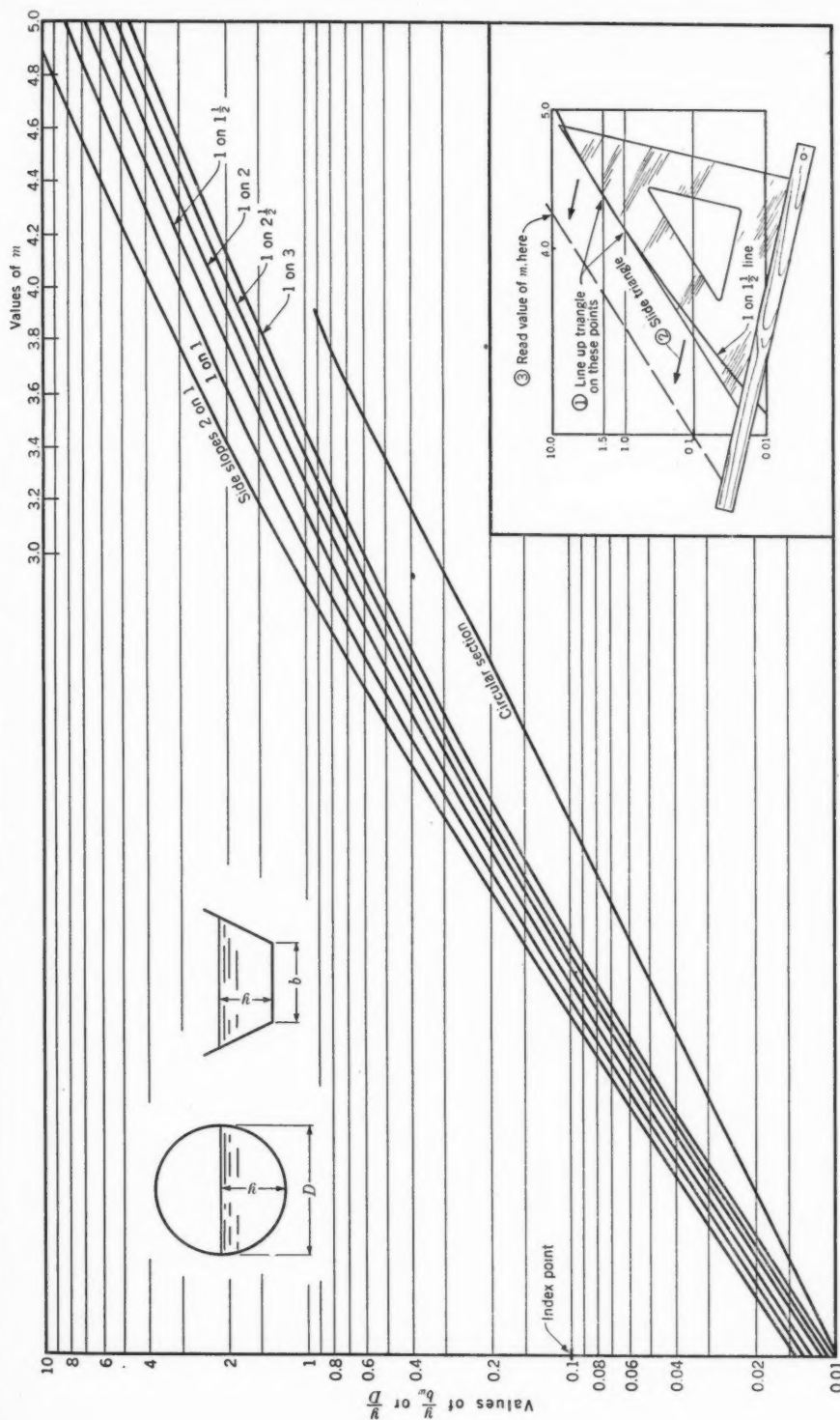
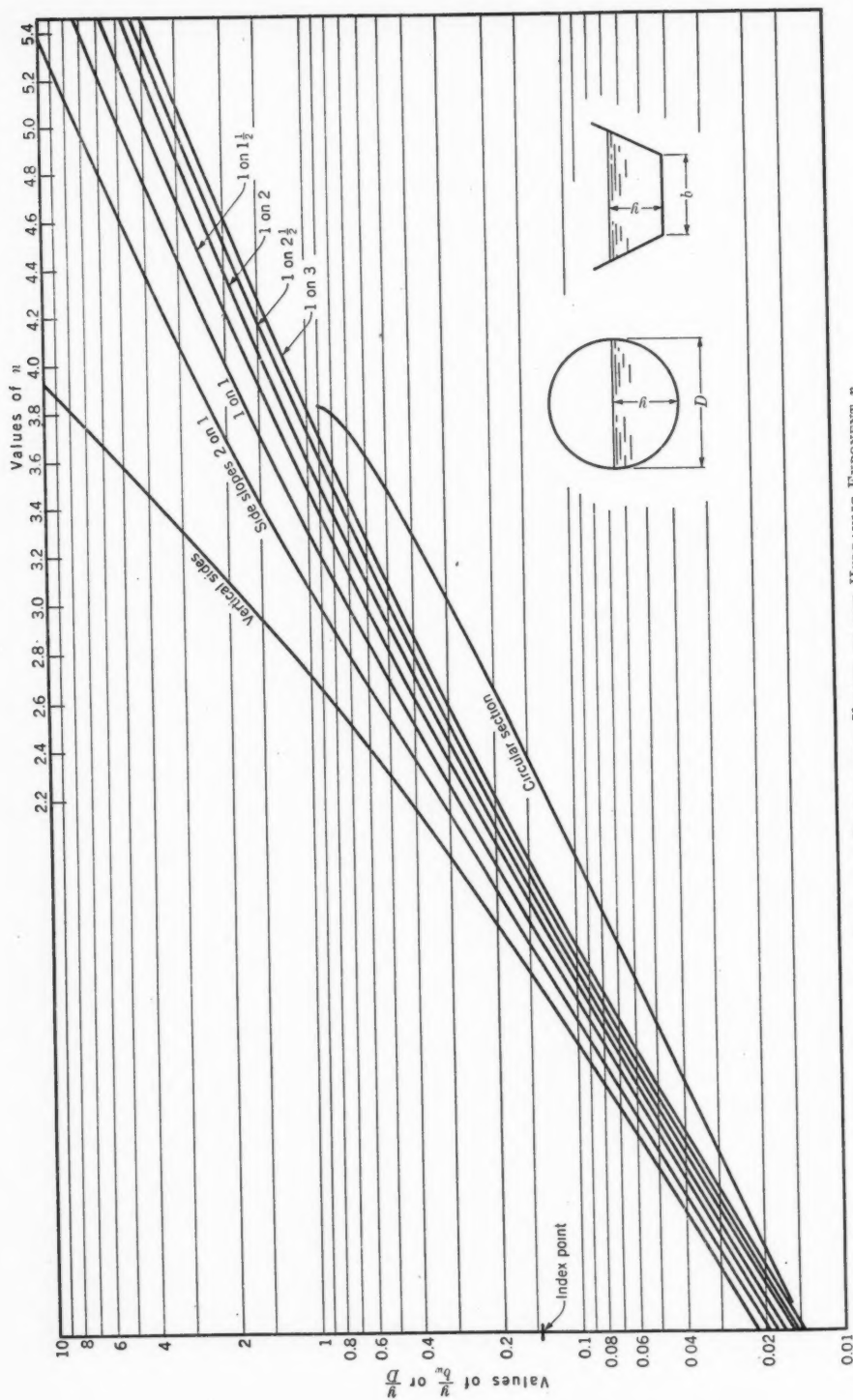


FIG. 1.—CHART FOR DETERMINING VALUES OF THE HYDRAULIC EXPONENT  $m$

FIG. 2.—CHART FOR DETERMINING VALUES OF THE HYDRAULIC EXPONENT  $n$

edge slide the triangle to the left until it coincides with the index point at the left of the chart. Read the value of  $m$  where the triangle intersects the scale at the top of the chart. In this case, the value of  $m$  is slightly less than 4.2. For ordinary computations the nearest 0.1 will be accurate enough.

The value of  $n$  can be determined from a chart in a manner similar to the determination of  $m$ . Any of the flow formulas which can be reduced to the form:

$$Q = C A R^a S^b \dots \dots \dots (18)$$

may be used, provided that  $C$  contains only factors pertaining to the shape and to the roughness of the channel and that the roughness coefficient is considered constant throughout the cross section. The Kutter formula is not applicable because in that case  $C$  contains the slope.

In the Manning formula,

$$K = (1.486/n) A R^{\frac{4}{3}} \dots \dots \dots (19)$$

so that

$$\left(\frac{y_1}{y_2}\right)^n = \left(\frac{K_1}{K_2}\right)^2 = \frac{[(1.486/n) A_1 R_1^{\frac{4}{3}}]^2}{[(1.486/n) A_2 R_2^{\frac{4}{3}}]^2} = \frac{(A_1 R_1^{\frac{4}{3}})^2}{(A_2 R_2^{\frac{4}{3}})^2} \dots \dots \dots (20)$$

in which  $n$ , when used with the constant 1.486, is the well-known Manning coefficient of roughness. Thus, the roughness coefficient cancels out and the hydraulic exponent  $n$  is a function of the shape of the channel. In Fig. 2 are plotted values of  $\log y$  against values of  $\log K$  for a unit bottom width and for different side slopes. The chart is used exactly as is Fig. 1.

#### COMPUTATION PROCEDURE

To illustrate the use of the method in nonuniform flow problems, the following typical examples are shown:

**Example 1 ( $M_1$ -Curve).**—In a channel (Fig. 3) with a discharge of 800 cu ft per sec, and with Manning's  $n = 0.025$  and  $y_2 = 8.0$  ft, it is required to determine the length of the curve. For  $Q = 800$  and  $S_0 = 0.0004$ ,  $y_0 = 6.73$  ft

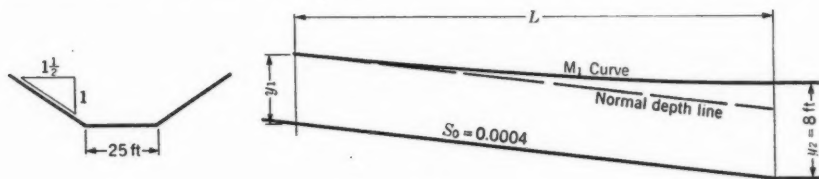


FIG. 3

and  $y_c = 2.98$  ft. Because  $y_0 > y_c$ , and the whole curve lies above the normal depth line, this is an  $M_1$ -curve. The values of  $n$  and  $m$  are obtained by dividing 8.0 ft and 2.98 ft, the values of  $y_2$  and  $y_c$ , respectively, by the bottom width of the channel—25 ft. This procedure gives 0.32 and 0.119, respectively; and, from Figs. 1 and 2,  $m = 3.4$  and  $n = 3.6$ , using the 1 on  $1\frac{1}{2}$  slope line in each case.

Theoretically the surface curve would be infinitely long, but it is impractical to extend it past the point where  $u = 0.999$ . The curve, then, will extend from where  $u_2 = 6.73/8.00 = 0.841$  to where  $u_1 = 0.999$ . From Eq. 13 in which  $y_0/S_0 = 16,825$  and from Eq. 12 in which  $(y_c/y_0)^n = 0.063$ , the following values are found:

$y$	$u$	$Y$	$(\frac{y_c}{y_0})^m Y$	$Z$	$\Pi$	$\Delta\Pi$
8.00	0.841	0.368	0.023	0.857	0.880	1.493
6.74	0.999	1.769	0.111	-0.724	-0.613	

—from which  $L = 25,100$  ft. If it is desired to know what the stage would be at a distance of 5,000 ft upstream from  $y_2$  by Eq. 13 it is found that  $\Pi_1 = 0.880 - 0.297 = 0.583$ .

The value of  $u$  which will make  $\Pi_1$  equal to 0.583 is found by trial and error, as follows:

$u$	$Y_1$	$(\frac{y_c}{y_0})^m Y_1$	$Z_1$	$\Pi_1$	$y_1$
0.92	0.554	0.035	0.572	0.607	....
0.93	0.591	0.037	0.525	0.562	....
0.925		(interpolated)		0.583	7.29

**Example 2 ( $M_2$ -Curve).**—A channel with 1 on 1 side slopes and  $S_0 = 0.0003$  ends in an abrupt drop (Fig. 4). It is desired to determine the length of the surface curve if  $Q = 800$  cu ft per sec and Manning's  $n = 0.030$ .

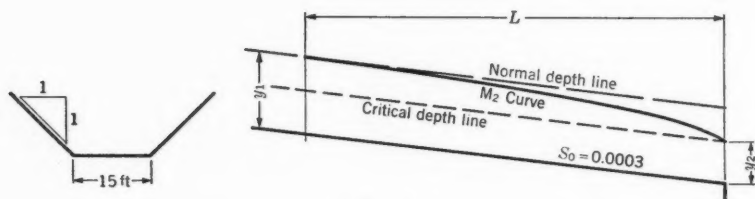


FIG. 4

With the conditions given,  $y_0 = 10.8$  ft,  $y_c = 4.05$  ft,  $y_0/S_0 = 10.8/0.0003 = 36,000$ , and  $(y_c/y_0)^m = (4.05/10.8)^{3.4} = 0.0356$ . Also,  $y_0/15 = 0.72$  and  $y_c/15 = 0.27$ ; and, from Figs. 1 and 2,  $m = 3.4$  and  $n = 3.6$ . In Eqs. 12 and 13 the values are found to be

$y$	$u$	$Y$	$(\frac{y_c}{y_0})^m Y$	$Z$	$\Pi$	$\Delta\Pi$
4.05	2.667	0.259	0.009	-0.002	0.007	1.384
10.79	1.001	2.283	0.081	-1.458	-1.377	

—from which  $L = 49,800$  ft.

**Example 3 ( $M_3$ -Curve).**—A sluice discharges into a rectangular channel laid on a slope of 0.0004;  $Q = 300$  cu ft per sec and Manning's  $n = 0.015$ . The depth at the vena contracta is 0.5 ft and a jump occurs where  $y_2 = 0.8$  ft (Fig. 5). It is required to determine the position of the jump.

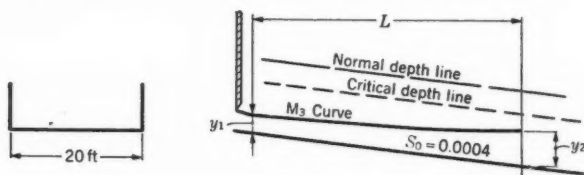


FIG. 5

The position of the jump will be determined by the length of the curve between  $y = 0.5$  and  $y = 0.8$ . For a rectangular channel  $m = 3.0$ , and by entering Fig. 2 with values of  $y/b$  of  $0.5/20 = 0.025$  and  $0.8/20 = 0.040$ , it is found that  $n = 3.4$ . Then  $y_0 = 3.83$  ft,  $y_c = 1.92$  ft,  $(y_c/y_0)^m = 0.126$ , and  $y_0/S_0 = 3.83/0.0004 = 9,575$ , from which the values in Eqs. 12 and 13 are found to be

$y$	$u$	$Y$	$(\frac{y_c}{y_0})^m Y$	$Z$	$\Pi$	$\Delta\Pi$
0.50	7.66	0.042	0.005	0	0.005	0.005
0.80	4.79	0.080	0.010	0	0.010	

—from which  $L = 48$  ft.

**Example 4 ( $S_1$ -Curve).**—A channel with 1 on  $\frac{1}{2}$  side slopes and a bottom width of 15 ft carries 1,000 cu ft per sec and empties into a pool so that  $y_2$ , the depth at the pool, is 10 ft (Fig. 6). If  $y_1$ , the depth after the jump, is 6.2 ft, locate the position of the jump ( $S_0 = 0.01$  and Manning's  $n = 0.020$ ).

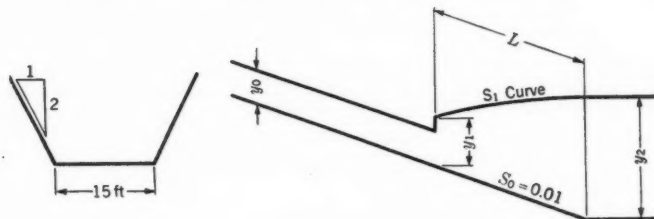


FIG. 6

The first step is to determine  $y_0$  and  $y_c$ , which are found to be 3.97 ft and 4.93 ft, respectively. From Figs. 1 and 2, it is found that  $m = 3.2$  and  $n = 3.2$ . Then,  $y_0/S_0 = 397$  and  $(y_c/y_0)^m = 1.99$ ; and, in Eqs. 12 and 13, the following values obtain:

$y$	$u$	$Y$	$(\frac{y_c}{y_0})^m Y$	$Z$	$\Pi$	$\Delta\Pi$
10.0	0.397	0.061	0.121	2.459	2.580	0.828
6.2	0.640	0.190	0.379	1.373	1.752	

—from which  $L = 328$  ft.

**Example 5 ( $S_2$ -Curve).**—Fig. 7 shows a typical case of this type of curve, for which it is required to determine the length ( $Q = 1,000$  cu ft per sec, Manning's  $n = 0.013$ , and  $S_0 = 0.005$ ).

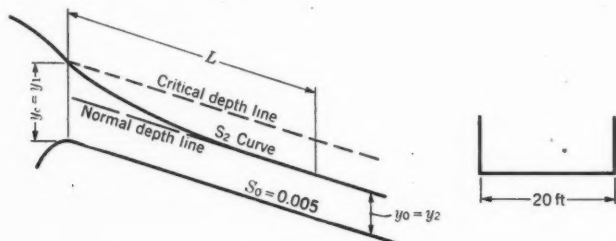


FIG. 7

Computations give  $y_0 = 3.35$  ft and  $y_c = 4.27$  ft. For this range of depths, the hydraulic exponents,  $n$  and  $m$ , both have a value of 3.0. Therefore, by Eqs. 12 and 13 the following values are found:  $y_0/S_0 = 670$ ,  $(y_c/y_0)^m = 2.073$ , and

$y$	$u$	$Y$	$(\frac{y_c}{y_0})^m Y$	$Z$	$\Pi$	$\Delta\Pi$
4.27	0.785	0.396	0.821	0.877	1.698	1.645
3.353	0.999	2.183	4.525	-1.182	3.343	

—from which  $L = 1,102$  ft.

**Example 6 ( $S_3$ -Curve).**—A sluice discharges into a rectangular channel on a slope of 0.008;  $Q = 600$  cu ft per sec, Manning's  $n = 0.015$ , and the depth at the vena contracta is 1.6 ft. Determine the length of the surface curve (Fig. 8). In Eqs. 12 and 13,  $y_0 = 2.25$  ft,  $y_c = 3.03$  ft,  $y_0/S_0 = 281$ ,  $(y_c/y_0)^m = 2.444$ ,  $m = 3.0$ ,  $n = 3.0$ , and the following values are found:

$y$	$u$	$Y$	$(\frac{y_c}{y_0})^m Y$	$Z$	$\Pi$	$\Delta\Pi$
1.60	1.406	0.793	1.938	-0.081	1.857	3.168
2.248	1.001	2.788	6.814	-1.789	5.025	

—from which  $L = 890$  ft.

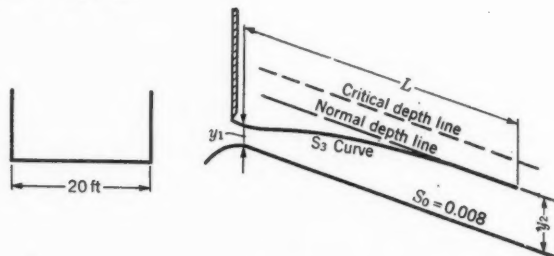


FIG. 8

**Example 7 (Delivery Curve).**—This problem is included to show how easily and quickly the integration method arrives at a solution, whereas the use of a step method would be extremely cumbersome.

A canal 10,000 ft long with 1 on  $1\frac{1}{2}$  side slopes (Fig. 9) connects reservoir A with reservoir B, Manning's  $n = 0.025$  and  $S_0 = 0.0004$ . The pool at A is constant, but the pool at B fluctuates. The problem is to prepare a chart that will show the discharge through the canal for any level in pool B. Head lost at the entrance to the canal will be neglected to emphasize the nonuniform flow part of the problem.

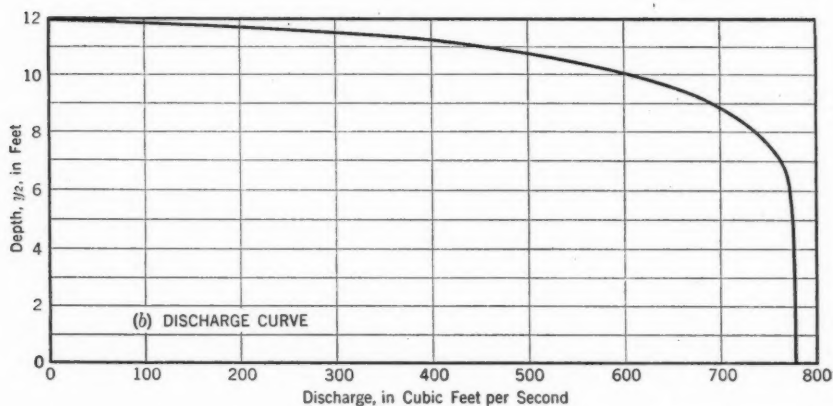
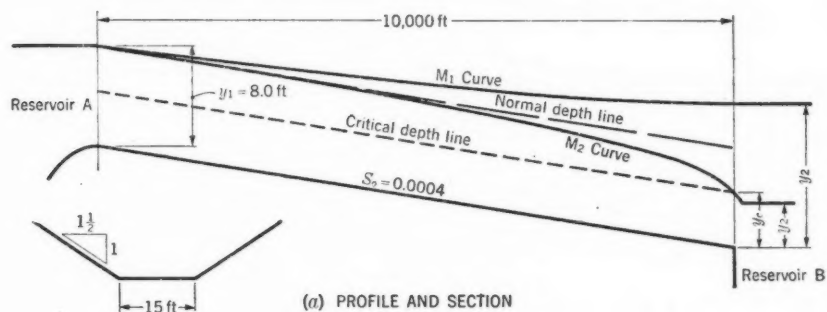


FIG. 9

Obviously, when the level at B is at the same elevation as that at A the discharge will be zero. As the pool level at B lowers, the discharge will increase and the surface curve will be of the  $M_1$ -type until the pool level is lowered to the point where it coincides with the normal depth line at the lower end of the canal (in other words to where  $y_2 = y_0$ ). Below this point flow is of the  $M_2$ -type and maximum discharge is reached when the pool at B is at or below the level of the critical depth line at the lower end of the canal.

When pool B is at the same level at pool A,  $Q = 0$ . Also, when  $y_2 = y_0$ , uniform flow obtains; and, by the Manning formula,  $Q = 744$  cu ft per sec. For intermediate points the flow is varied and the value of  $y_2$  corresponding to any selected value of  $Q$  is computed by Eqs. 12 and 13. By selecting values

of  $Q$ , the corresponding values of other quantities for values of  $m = 3.4$  and  $n = 3.6$  may be tabulated thus:

$Q$	$y_0$	$\frac{S_0}{y_0} L$	$y_c$	$(\frac{y_c}{y_0})^m$	$u_1$	$Y_1$	$(\frac{y_c}{y_0})^m Y_1$	$Z_1$	$\Pi_1$	$\Pi_2$
200	3.99	1.003	1.67	0.051	0.499	0.082	0.004	1.938	1.942	2.945
400	5.81	0.688	2.56	0.061	0.726	0.224	0.014	1.182	1.196	1.884
600	7.16	0.559	3.28	0.070	0.895	0.480	0.034	0.676	0.710	1.269
700	7.77	0.515	3.60	0.073	0.971	0.835	0.061	0.238	0.299	0.814

Values of  $u_2$  and  $y_2$  corresponding with the values of  $\Pi_2$  are found by trial and error. In this case, where the  $Y$ -function is not prominent, a close approximation can be made by referring to the tables of the  $Z$ -function. Two points are selected, the value of  $\Pi_2$  is computed for each, and the correct value of  $u_2$  is found by interpolation. Thus, for  $Q = 200$  cu ft per sec and  $\Pi_2 = 2.945$ ,

$u_2$	$Y_2$	$(\frac{y_c}{y_0})^m Y_2$	$Z_2$	$\Pi_2$	$y_2$
0.32	0.027	0.001	3.105	3.106	....
0.34	0.032	0.002	2.918	2.920	....
0.3385		(interpolated)		2.945	11.79

Similarly, for  $Q = 400$  cu ft per sec,  $y_2 = 11.33$  ft; for  $Q = 600$  cu ft per sec,  $y_2 = 10.23$  ft; and, for  $Q = 700$  cu ft per sec,  $y_2 = 9.00$  ft.

The maximum discharge is determined by transposing Eq. 13 and solving for  $\Pi_1$ . Maximum discharge will occur if  $y_2 = y_c$  (see Fig. 9(a)). By taking a series of discharges from 744 cu ft per sec upward and setting  $y_2 = y_c$ , the corresponding values of  $y_1$  are determined. The value of  $Q$  for which  $y_1 = 8.0$  ft is the maximum  $Q$ . Again,  $m = 3.4$  and  $n = 3.6$ . Thus,

$Q$	$y_0$	$y_c$	$\frac{S_0}{y_0} L$	$(\frac{y_c}{y_0})^m$	$u_2$	$Y_2$	$(\frac{y_c}{y_0})^m Y_2$	$Z_2$	$\Pi_2$	$\Pi_1$
770	8.14	3.80	0.491	0.075	2.142	0.340	0.025	-0.007	0.018	-0.473
800	8.30	3.89	0.482	0.077	2.134	0.341	0.026	-0.007	0.019	-0.463

To determine the value of  $y_1$  that corresponds to  $\Pi_1$ , trial values of  $u_1$  are selected and trial values of  $\Pi_1$  are computed. The correct values of  $u_1$  are found by interpolation:

	$u_1$	$Y_1$	$(\frac{y_c}{y_0})^m Y_1$	$Z_1$	$\Pi_1$	$y_1$
For $\Pi_1 = -0.473$	1.02	1.450	0.109	-0.643	-0.534	....
	1.03	1.337	0.100	-0.539	-0.439	....
	1.026		(interpolated)		-0.473	7.92
For $\Pi_1 = -0.463$	1.02	1.450	0.112	-0.643	-0.531	....
	1.03	1.337	0.103	-0.539	-0.436	....
	1.027		(interpolated)		-0.463	8.07

Interpolating between these two values of  $y_1$  gives  $Q_{\max} = 786$  cu ft per sec and  $y_2 = 3.85$  ft. If one point is computed between  $Q = 744$  and  $Q = 786$ —

say, at  $Q = 772$ —the values in Eqs. 12 and 13 are found to be

$Q$	$y_0$	$\frac{S_0}{y_0} L$	$y_c$	$(\frac{y_c}{y_0})^m$	$u_1$	$Y_1$	$(\frac{y_c}{y_0})^m Y_1$	$Z_1$	$\Pi_1$	$\Pi_2$
772	8.15	0.491	3.80	0.075	1.02	1.450	0.109	-0.643	-0.534	-0.043

from which computation by trial and error gives  $y_2 = 6.43$  ft. The computed points are plotted and the delivery curve is drawn (Fig. 9(b)).

### CONCLUSIONS

The average engineer thinks in terms of uniform flow, which for simple problems may give a sufficient answer. In the ordinary canal, however, non-uniform flow is encountered as often as uniform flow, if not more often. Therefore, it is imperative that the engineer learn to think in terms of nonuniform flow. It is important, also, that he have a convenient method of working with nonuniform flow.

An integration method such as the one presented shortens the time required to compute backwater problems. What is probably more important is that such a method makes it possible, successfully and quickly, to attack nonuniform flow problems that are seldom tried by ordinary step methods.

The method presented can be used with the Kutter, the Manning, the Bazin, or any other of the open channel formulas used to compute uniform flow. It can also be applied to natural streams, to channels with horizontal bottom, and to channels of adverse slope, although with the last a special set of tables of the  $Y$ -function and the  $Z$ -function would have to be computed.

### ACKNOWLEDGMENT

The writer wishes to express his appreciation to Professor Posey of the University of Iowa at Iowa City for his review of this paper and for his words of encouragement.

### APPENDIX I. NOTATION

The following letter symbols, defined where they first appear in the text, or by illustration, conform essentially with the American Standard Letter Symbols for Hydraulics (ASA—Z10.2—1942) prepared by a Committee of the American Standards Association, with Society representation:

- $A$  = cross-sectional area;
- $b$  = width of stream bed;  $b_w$  = width of water surface;
- $C$  = the Chézy coefficient of flow; also, a constant of integration;
- $D$  = diameter of circular section;
- $e$  = base of Napierian logarithms;
- $g$  = gravitational acceleration;
- $K$  = cross-sectional conveyance (defined by Eq. 2a);  $K_0$  = cross-sectional conveyance for uniform flow;
- $L$  = length of surface curve;

- $M$  = a cross-sectional function (defined by Eq. 3a);  $M_c$  = a cross-sectional function for the critical depth (defined by Eq. 3b);  
 $m$  = an hydraulic exponent (defined by Eq. 5b);  
 $n$  = an hydraulic exponent (defined by Eq. 5a); also the Manning coefficient;  
 $p$  = a substitution factor (defined by Eq. 24);  
 $Q$  = discharge (rate of flow);  
 $R$  = hydraulic radius of cross-sectional area;  
 $S$  = slope:  
      $S_f$  = slope of energy grade line (friction slope);  
      $S_s$  = slope of water surface;  
      $S_0$  = slope of channel bed (also slope of water surface for uniform flow);  
 $u$  = a substitution factor (defined by Eq. 7a);  
 $V$  = velocity of flow;  
 $v$  = a substitution factor (defined by Eq. 7b);  
 $x$  = a substitution factor (defined by Eq. 22); also distance along canal parallel to bottom grade as used in Eqs. 1 to 11;  
 $Y$  = a substitution factor (defined by Eq. 9a);  
 $y$  = depth of flow:  
      $y_c$  = critical depth;  
      $y_0$  = depth for uniform flow;  
 $Z$  = a substitution factor (defined by Eq. 9b); and  
 $\Pi$  = a substitution factor (defined by Eq. 12).

## APPENDIX II. COMPUTATION OF TABLES OF Y-FUNCTIONS AND Z-FUNCTIONS

*Values of  $u$  Less Than 1.0.*—Eq. 9a can be written as

$$Y = \int (u^{m-2} + u^{n+m-2} + u^{2n+m-2} + u^{3n+m-2} + \dots) du \dots (21a)$$

from which

$$Y = \frac{u^{m-1}}{m-1} + \frac{u^{n+m-1}}{n+m-1} + \frac{u^{2n+m-1}}{2n+m-1} + \frac{u^{3n+m-1}}{3n+m-1} + \dots (21b)$$

The series converges for  $u$  less than 1.0, and for small values of  $u$  the convergence is rapid. Thus, only a few terms need be computed in each series. As  $u$  becomes large, more terms are required so that at the upper range of values (from  $u = 0.90$  to  $u = 0.999$ ) it is convenient to transpose the integral. In Eq. 9a assume that

$$1 - u^n = x \dots (22)$$

so that Eq. 9a reduces to

$$Y = \int \frac{(1-x)^p}{x} dx \dots (23)$$

in which

$$p = \frac{m - n - 1}{n} \dots \dots \dots (24)$$

Expanding and integrating,

$$Y = -\frac{1}{n} \left[ C + \log_e x - p x + \frac{p(p-1)x^2}{2 \times 2!} - \frac{p(p-1)(p-2)x^3}{3 \times 3!} + \dots \right] \dots (25)$$

in which  $C$  is a constant of integration and  $e$  is the base of the Napierian logarithms. This expression converges more rapidly as  $u$  becomes larger.

For the range between  $u = 0.70$  and  $u = 0.90$ , the tables were computed by differentials, the slope of the curve of  $Y$  versus  $u$  being

$$\frac{dY}{du} = \frac{u^{m-2}}{1 - u^n} \dots \dots \dots (26)$$

The difference between the values of  $Y$  for  $u_1$  and  $u_2$  is  $(u_2 - u_1)$  multiplied by the average slope of the curve between the two points. The interval  $u_2 - u_1$  must be small enough to bring this approximation within the limits of accuracy desired.

To compute the  $Z$ -table Eq. 9b is expanded and integrated to give

$$Z = \frac{1}{u} - \frac{u^{n-1}}{n-1} - \frac{u^{2n-1}}{2n-1} - \frac{u^{3n-1}}{3n-1} - \dots \dots \dots (27a)$$

It will be noted that, if  $n = m$ , Eq. 27a can be written as

$$Z = \frac{1}{u} - Y \dots \dots \dots (27b)$$

*Values of  $u$  Greater Than 1.0.*—To obtain a converging series for  $u$  greater than 1.0, let

$$u = 1/x \dots \dots \dots (28)$$

Then Eq. 9a becomes

$$Y = \int \frac{x^{n-m} dx}{1 - x^n} = \int (x^{n-m} + x^{2n-m} + x^{3n-m} + \dots) dx \dots \dots (29a)$$

or

$$Y = \frac{x^{n-m+1}}{n-m+1} + \frac{x^{2n-m+1}}{2n-m+1} + \frac{x^{3n-m+1}}{3n-m+1} + \dots \dots \dots (29b)$$

Eq. 29b was used for values of  $u$  from 1.10 to 30.0.

For values of  $u$  between 1.001 and 1.10, the number of terms in the series becomes impractical so that it is convenient to transpose the equation. Thus, by letting

$$-x = 1 - u^n \dots \dots \dots (30)$$

and, by substituting in Eq. 9a,

$$Y = -\frac{1}{n} \left[ C + \log_e x + p x + \frac{p(p-1)x^2}{2 \times 2!} + \frac{p(p-1)(p-2)x^3}{3 \times 3!} + \dots \right] \dots (31)$$

The series represented by Eq. 31 converges more rapidly as  $u$  approaches 1.0.  
By substituting Eq. 28 in Eq. 9b,

$$Z = - \int \frac{x^n dx}{1 - x^n} \dots \dots \dots (32a)$$

which, when expanded and integrated, gives

$$Z = - \frac{x^{n+1}}{n+1} - \frac{x^{2n+1}}{2n+1} - \frac{x^{3n+1}}{3n+1} - \dots \dots \dots (32b)$$

All values of the  $Z$ -function are negative for values of  $u$  greater than 1.0, and it will be noted that, where  $n = m$ ,

$$Z'_1 = x - Y \dots \dots \dots (33)$$

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# AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

## PAPERS

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### CADASTRAL SURVEYS AND RESURVEYS

BY A. C. HORTON, JR.<sup>1</sup>

#### INTRODUCTION

In accordance with Reorganization Plan No. 3 of President Harry S. Truman, in 1946, the Bureau of Land Management (hereinafter called "the Bureau") was established in the United States Department of the Interior on July 16, 1946, through consolidation of the General Land Office and the Grazing Service. The General Land Office was established in 1812 and the Grazing Service in 1934. By the consolidation, the administration of approximately 400,000,000 acres of public land in the west was brought into one unit to provide for more effective administration at less cost and to insure maximum service in the public interest by the decentralization of activities.

In addition to administering the Taylor Grazing Act of 1934, the Bureau exercises a large measure of primary responsibility in the administration of the public land laws. Its duties range from the identification and description of the public lands by cadastral surveys and resurveys to providing for their acquisition or use in the development of mineral resources—in addition to the disposal of lands under the homestead, townsite, and other laws; the leasing of land; the granting of rights of way; the classification of lands; the adjudication of applications from the standpoint of law and conservation policy; and the many other duties placed upon it by public land statutes.

A reference here to the origin of certain duties of the Bureau should reveal more clearly the true significance of cadastral surveys, records, and legal descriptions and the part they have played and will play in the development of the United States.

#### HISTORY

In 1784 the Continental Congress appointed a committee headed by Thomas Jefferson to draft "an ordinance for ascertaining the mode of locating and disposing of lands in the western territory." The rectangular system of surveying public lands was inaugurated under the ordinance passed by the Continental Congress on May 20, 1785. In brief, the ordinance provided for townships 6 miles square, each containing 36 sections of 1 mile square, "as near as may be." With the location of base lines, standard parallels, and

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NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by June 1, 1949.

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principal and guide meridians (for offsetting the effects of convergency) and the designation of townships, ranges, and sections, it became possible to identify conveniently and with certainty any tract of land anywhere, and to give it a place on a map. The public land area—including, at its maximum extent, the Territory of Alaska, the states of Florida, Alabama, and Mississippi and all states except Texas lying north and west of the Ohio and Mississippi rivers—was subject to survey under this rectangular plan, admittedly the most convenient and simple scheme ever devised by any nation.

Numerous refinements of practice and improvements of methods of far-reaching importance have been made a part of the rectangular system since its beginning. It has been shaped to meet newer aspects of the older problems and has been developed to take care of the intricate practice of resurveying which time and the elements and the frailty of human nature have made necessary; but in principle and in purpose it is the same now as it was then, and is simply the rectangular system of coordinates applied in a practical manner. Rectangular coordinates recognize north in its true position and verify rather than refute Galileo in the age-old theory that the world is round.

Since 1785 the extension of the rectangular system of surveys over more than 2,000,000 sq miles of public land area has created a vast monumented cadastral net. Each square mile is monumented at its four corners, with a monument halfway down each of its four sides in order that crosslines may be run establishing the boundaries of quarter sections. The monuments are the corners for which they were established, even though not exactly where professional care might have placed them in the first instance. Missing corners must be re-established in the identical positions they originally occupied, and when the positions cannot be determined by existing monuments or other verifying evidence, resort must be had to the field notes of the original survey. Law provides that the lengths of the lines as returned in the field notes shall be held as the true lengths, and the distances between identified corner positions given in the field notes constitute proper data from which to determine the position of a lost corner. Hence, the logical and equitable rule is that lost corners are restored at distances proportionate to the original measurements between identified positions.

Since 1910 when the contract system for executing surveys was abolished, the corners of public land surveys have been monumented with a standard metal corner post manufactured from wrought iron or copper bearing steel pipe, filled with a core of concrete. A brass cap riveted to one end of the post is appropriately marked with steel dies. The planted end of the pipe is split and the two halves are spread to form a flange.

#### SCOPE

The rectangular system of surveys has been extended over approximately 92% of the total area of the public lands in the continental United States; but it has covered less than 1% of the area of the Territory of Alaska. The area remaining unsurveyed in the continental United States—comprising approximately 117,150,000 acres—is equal to about the combined area of the states of Arizona and Washington. However, the virgin cadastral survey field of large

extent, amounting to about 567,350 sq miles under the jurisdiction of the Bureau, is in Alaska. The resulting vast and comprehensive rectangular survey net provides a simple and certain form of land identification, and anchors in place the land of the public domain.

The rectangular cadastral survey is a prerequisite to the administration and conveyance of the public lands. Under it, area is identified by fixed position on the ground and by descriptive data in the records. Cadastral surveys, unlike scientific surveys of an information character which may be amended with changing conditions or because of error, mark for all time the boundaries of lands on the earth's surface. Early public land surveys may be physically lost, or in error, or fraudulent in whole or in part, but they still define boundaries and area for all time with descriptive fixity of place. In the execution of cadastral resurveys it is the cadastral engineer's problem at times to restore, legally, something that never existed on the ground and to preserve and protect all valid existing rights in the process.

All land titles, land transactions, and considerations of geographical position in the area of the public domain rest upon and are in terms of the Bureau's cadastral public land survey, and its basic, legal descriptions should not be duplicated, confused, and obscured unnecessarily by supplemental records of survey systems alien in character. Cadastral surveys are fundamental factors in the formulation and operation of laws and policies relating to the public lands and, therefore, in a very definite sense, in the orderly administration of the natural resources.

The Bureau is the only congressionally constituted agency charged with the execution of cadastral surveys of the public lands, but the fact is not always appreciated that these surveys do not ascertain boundaries—but instead create them. Cadastral surveys, in contradistinction to information and construction surveys, are made primarily for jurisdictional and proprietary purposes.

Cadastral surveys in the process of execution are divided into two major procedures—namely, the creation and marking of the survey lines upon the ground and the preparation of the official record. The additional step of approval and acceptance by competent authority is necessary under the law to give such surveys their official and legal standing.

The final survey record consists of field notes describing in detail the processes of the survey, the results on the ground, and the plat which is a graphic representation of the surveys made in the field. The primary purpose of the plat is to show the courses and lengths of survey lines and, incidentally, relief, drainage, and culture. Its basic function is to designate and describe areas in specific terms and to serve as the legal basis for all transactions involving the public lands within its borders. The legal significance of the plat in this respect is as important as it would be if incorporated into the patent itself and the same applies to any subsequent deed or transfer.

Although incidental to the primary purpose and basic function of the cadastral survey plat, an adequate and accurate portrayal of relief, drainage, and culture on present and future plats is essential to the proper administration and management of the public lands in conserving natural resources. To meet this demand, the cadastral engineering organization has adopted the

use of aerial photographs, resulting in a comprehensive and accurate depiction of relief, drainage, and culture on the cadastral plat.

By application of the system of radial plot, section corners that can be identified on the photographs are selected for ground control. Other corners are resected to the photographs, thereby correlating the lines of the ground survey with the aerial survey. A sufficient number of topography points are selected, and the planimetry is then transferred from the photographs to the plat.

Relief is determined through the measurement of parallax. Vertical control is obtained from clinometer readings taken at the time of survey and tied in, where available, to official bench marks. By measuring parallax, additional spot elevations are found on the photographs and relief is sketched in and transferred to the plat. Normally, the hachure is used to depict relief.

Although the rectangular system remains the basic principle of the modern cadastral survey, the regular extension of townships and ranges is no longer the principal activity. Except in restricted areas within the public land states and in the Territory of Alaska, cadastral engineers are no longer concerned primarily with the mere determination of the length and direction of lines and with the establishment of corner monuments at specified intervals. Such determinations are made, it is true (as are those of time, latitude, and azimuth), with modern instrumental equipment unknown to the pioneer surveyor, and with a degree of precision and refinement of method not approached in early days. Today and in the future, however, the real problems confronting cadastral engineers are those of restoration and the preservation of vested rights.

Cadastral resurveys of large areas of the public domain are being conducted in accordance with the best engineering practice, and with strict attention to the legal and equitable considerations involved in the proper protection of vested rights. Grant and reservation boundaries, often practically obliterated, are being recovered, and their identity established beyond question. Important riparian boundaries which often control rights of great value, and of which practically no physical evidence remains, are being restored and adjusted. Reconnaissance and restoration resurveys are undertaken where information relative to the position of original boundaries is required as evidence in issues pending before the courts.

The cooperative program of cadastral surveys and resurveys, as requested by the various federal bureaus and departments aggregates 110,000,000 acres, and includes public lands under the jurisdiction or administration of the Bureau of Indian Affairs (United States Department of the Interior), the Forest Service (United States Department of Agriculture), the Bureau of Reclamation (United States Department of the Interior), the Geological Survey (United States Department of the Interior), the National Park Service (United States Department of the Interior), the Federal Power Commission, and others. These bureaus require carefully executed and permanently monumented cadastral surveys for the identification, classification, and administration of the lands. The agencies engaged in mapping these areas find that the cadastral surveys furnish excellent horizontal control for topographic mapping purposes—particularly where aerial photography is being used. An example of a cooperative resurvey follows.

In addition to its regular operations throughout the continental United States and Alaska, the cadastral engineering organization of the Bureau is now (1948) engaged in the identification of more than 10,000 sq miles of land areas for the Bureau of Reclamation within the Missouri Basin Project. On the project cadastral resurveys (with resulting plats) are a prerequisite to planning, land classification, and construction work; to the leasing, disposal, or purchase of land; and to the further subdivision of areas when project units are opened for settlement. The original surveys of the area were made in early days when the prime importance of permanent monumentation of corner positions was not realized as it is today. The ravages of time and the elements, through the many years, have all but obliterated evidence of the wooden stakes and pits that were originally established to identify the corner positions. The practiced eye of the cadastral engineer is required to identify the vague evidence remaining, and his intimate knowledge of intricate resurvey procedure is needed to re-establish, with permanent iron posts, the network of original surveys in their original position, whereby boundary lines between public and private ownership are readily identified.

The early surveys in the Missouri River Basin were made without the scientific instruments and modern methods in use today, and consequently the length and direction of boundary lines as determined by cadastral engineers are found to vary from that recorded originally. Precise length and direction of section boundaries are required for horizontal control in mapping.

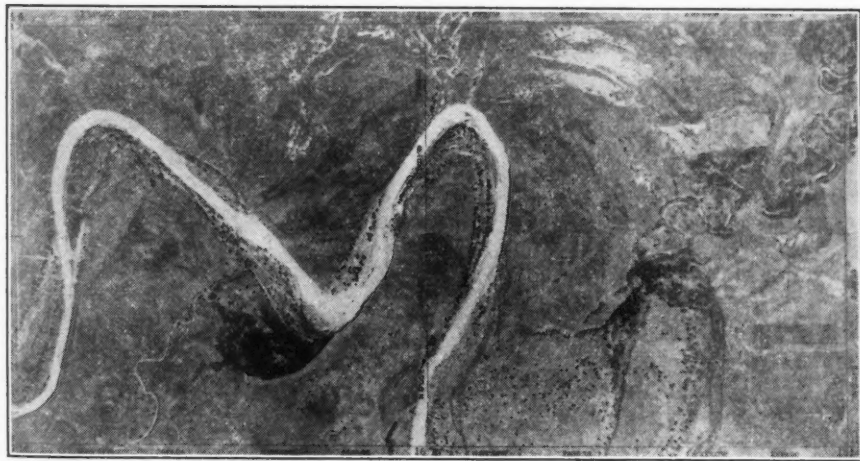


FIG. 1.

The basic need for land boundary identification and precise horizontal control in mapping operations on the Missouri Basin Project is illustrated by the aerial photograph in Fig. 1. The original is enlarged to a scale of 400 ft to the inch, and mounted on pressed wood, for use as a plane-table sheet. The photograph embraces sections 9 and 10, T 20 N, R 18 E, Black Hills Meridian, S. Dak., of the Grand River Unit, Missouri Basin Project. The area and

boundary lines of the two sections have been identified precisely on the aerial photograph by cadastral engineers of the Bureau. Topographic and soil classification plane-table parties of the Bureau of Reclamation use the aerial photographic sheet, covered with transparent vellum for the mapping requirements. A composite is made of the three, resulting in an accurate, final map. More than 10,000 sq miles of the Missouri Basin Project are to be relocated in the manner indicated by the exhibit.

The Bureau issues a number of publications dealing with the public land surveys and resurveys—the most important and comprehensive of which is the "Manual of Instructions for the Survey of the Public Lands of the United States." The "Standard Field Tables" and an annual publication of the "Ephemeris of the Sun, Polaris and other Selected Stars" are also available. A pamphlet entitled, "Restoration of Lost or Obliterated Corners and Subdivision of Sections," has been issued for the guidance of county and other local surveyors.

Copies of the plats and field notes representing all the public land surveys executed in the United States and in Alaska are on file in the Bureau office in Washington, D. C. The branch offices of the Bureau in the western states, such as the Public Survey Office at Phoenix, Ariz., have the survey records pertaining to the individual state in which they are located.

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## PAPERS

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### NOMOGRAPHIC ANALYSIS OF RECTANGULAR SECTIONS OF REINFORCED CONCRETE

BY W. P. LINTON,<sup>1</sup> M. ASCE

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#### SYNOPSIS

Several methods are in current use for the design of reinforcement in rectangular sections of reinforced concrete subjected to bending and direct stress. However, most of them require the use of numerous diagrams or tables; involve long and laborious computations; and are useful mainly in analyzing designs previously made, rather than in making an original design. In designing it is customary to assume the amount of reinforcement and then to check the assumption. If the assumed reinforcement is inadequate or uneconomical, a new assumption must be made and the process repeated until a satisfactory design is obtained. This process is laborious and time consuming. Many diagrams or tables are usually required, because, on account of the number of variables,  $d'/d$  is made constant and one diagram is drawn for each value of  $d'/d$ . A few values of  $d'/d$  are usually selected but in practice it is frequently found that the actual value of  $d'/d$  does not conform with any one of the diagrams. Then the diagram made for the nearest value of  $d'/d$  must be used, at the sacrifice of considerable accuracy, or interpolation must be made between two diagrams and that process is inconvenient. The value of  $n$  also is assumed to be constant; therefore, if a different value of  $n$  is used, another set of diagrams is required so that, for a complete and satisfactory solution with reasonable accuracy, many diagrams are required.

Also an infinite number of combinations of  $p$  and  $p'$  will satisfy the equations, but usually no provision is made for determining the most economical combination.

The method presented herein obviates these objections, and is believed to be more complete and satisfactory and more easily applied than any other known to the writer. Only three nomograms which contain six variables need be

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NOTE.—Written comments are invited for publication; to insure publication the last discussion should be submitted by June 1, 1949.

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used. The nomograms were made primarily for bending and direct stress, but they are equally useful for simple bending because simple bending is merely a special case of the more general one.

The nomograms and examples of their use are explained to make the method of procedure clear. Then the derivation of the formulas and the method of construction of the nomograms are given.

The accepted theory of reinforced concrete is adhered to throughout without modification and the symbols used are standard and well known.

#### EXPLANATION OF THE NOMOGRAMS AND EXAMPLES OF THEIR USE

Six variables are involved in the three nomograms,<sup>1a</sup> Figs. 1, 2, and 3—namely,  $p' n$ ,  $p n$ ,  $d'/d$ ,  $k$ , **B**, and **C** and their significance is made clear in Fig. 4, with the formulas that follow:

$$M = N e \dots \dots \dots (1a)$$

$$B = \frac{N e'}{f_c b d^2} \dots \dots \dots (1b)$$

and

$$C = \frac{N}{f_c b d} \left[ \frac{e'}{d} - \left( 1 - \frac{d'}{d} \right) \right] \dots \dots \dots (1c)$$

For simple bending

$$B = C = \frac{M}{f_c b d^2} \dots \dots \dots (2)$$

Fig. 1 is a nomogram that contains the variables **B**, **C**,  $d'/d$ , and  $k$  and gives the value of  $k$  which will make the sum of  $(p n + p' n)$  a minimum when **B**, **C**, and  $d'/d$  are known. Fig. 2, similarly, is a nomogram that contains the variables **B**,  $p' n$ ,  $d'/d$ , and  $k$  and gives the value of any one of the four when the other three are known. Fig. 3, likewise, is a nomogram that contains the variables **C**,  $p n$ ,  $d'/d$ , and  $k$  and gives the value of any one of the four when the other three are known. The equations on which Figs. 2 and 3 are based are given in Appendix I and a brief review of the construction of nomograms is presented in Appendix II.

In designing reinforced concrete sections, the allowable unit stresses  $f_c$  and  $f_s$ , the ratio of the modulus of elasticity of steel to that of concrete  $n$ , and the embedment of the steel  $d'$  are given in the specifications or are known. The dimensions  $b$  and  $d$  are fixed by other considerations; and, therefore, the problem in design is to determine the most economical or satisfactory ratios of tensile and compressive reinforcement to fulfil the given requirements.

In checking a design of the reinforcement which has previously been made,  $p' n$ ,  $p n$ ,  $d'/d$  and the dimensions  $b$  and  $d$  are known and the problem is to find  $f_c$  and  $f_s$ . Since  $f_c$  is unknown, the variables **B** and **C** are also unknown.

Because the procedures in the two cases (that is, designing and checking) are different, examples of both are given which will make both procedures clear.

<sup>1a</sup> Original full-scale prints of the three nomograms will be mailed on receipt of \$1.00.

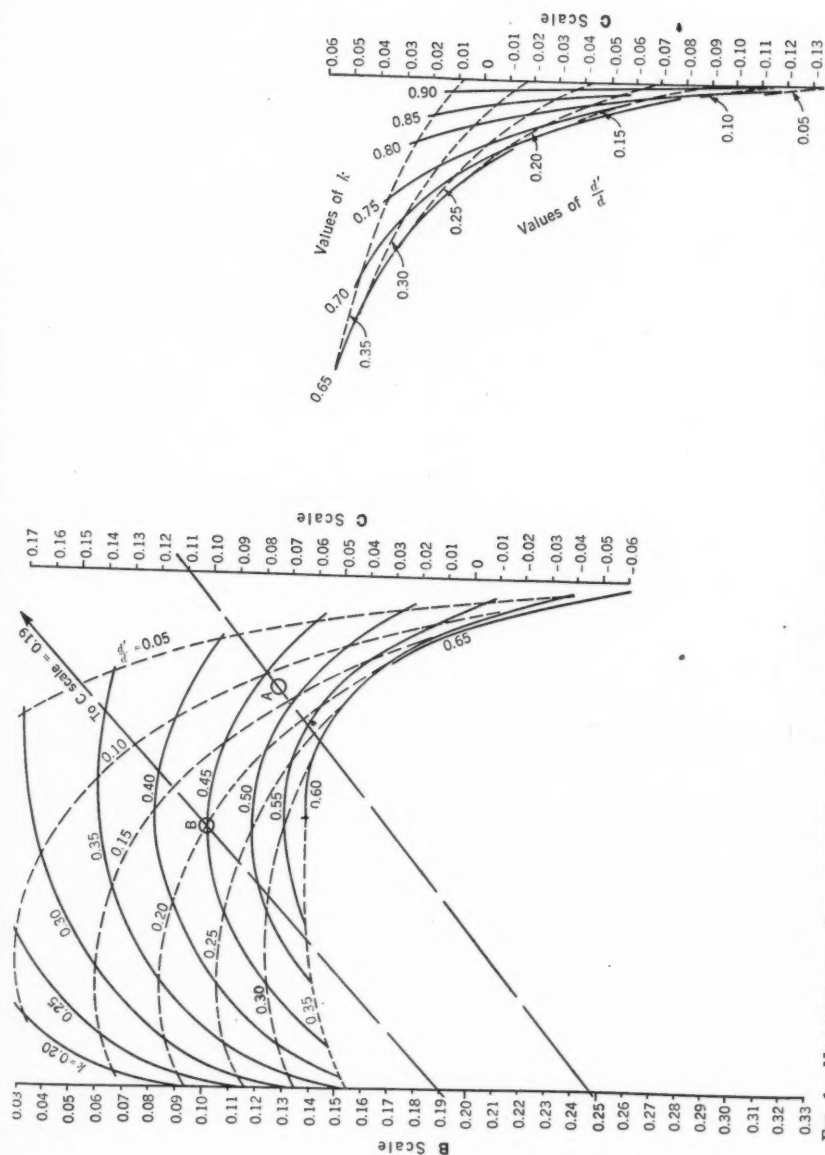
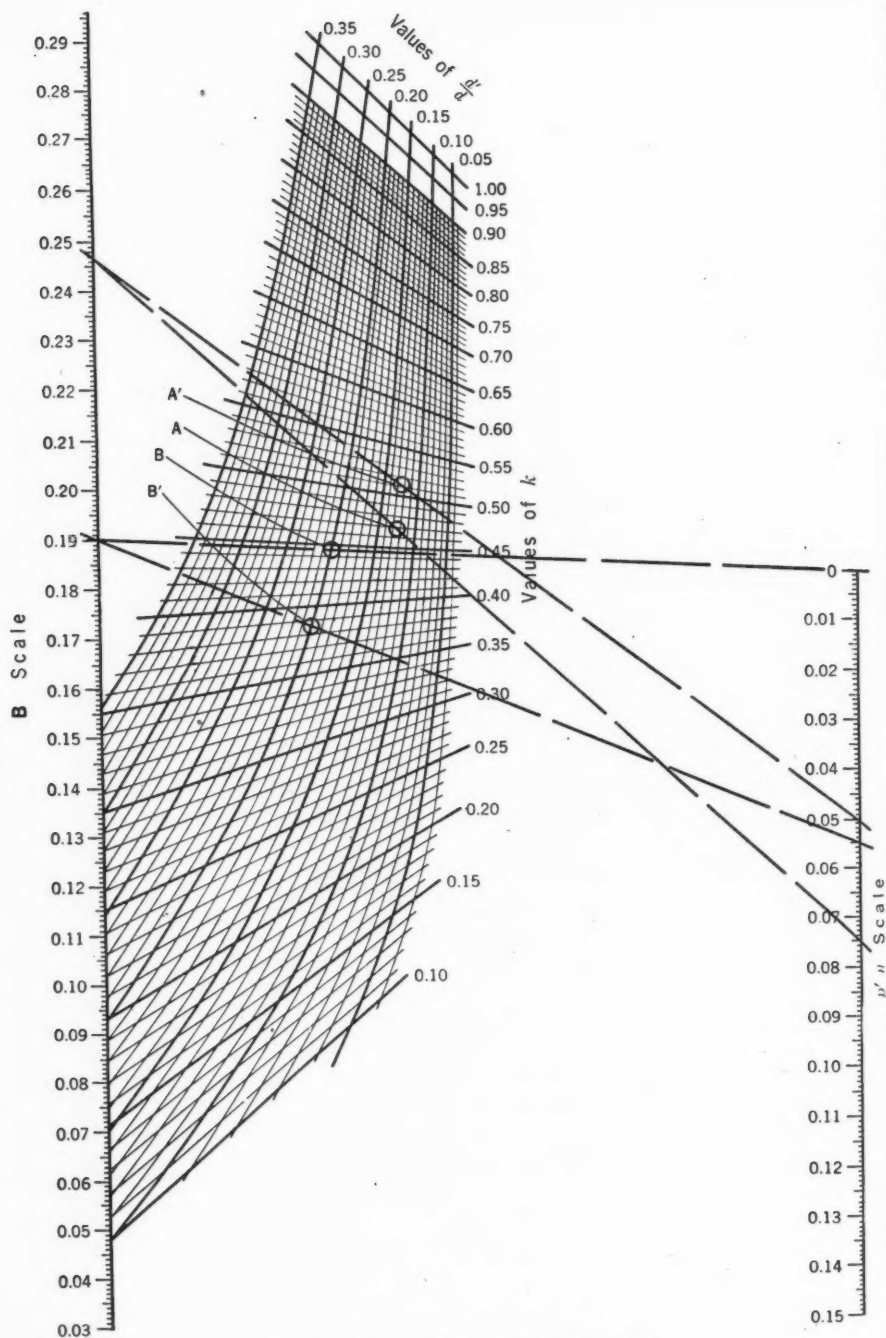


FIG. 1.—NOMOGRAM FOR FINDING THE VALUE OF  $k$  THAT WILL PRODUCE MINIMUM VALUES OF  $p_n + p' n$  FOR GIVEN VALUES OF  $B$ ,  $C$ , AND  $d'/d$

FIG. 2.—NOMOGRAM FOR  $B$ ,  $p'n$ ,  $d'/d$ , AND  $k$

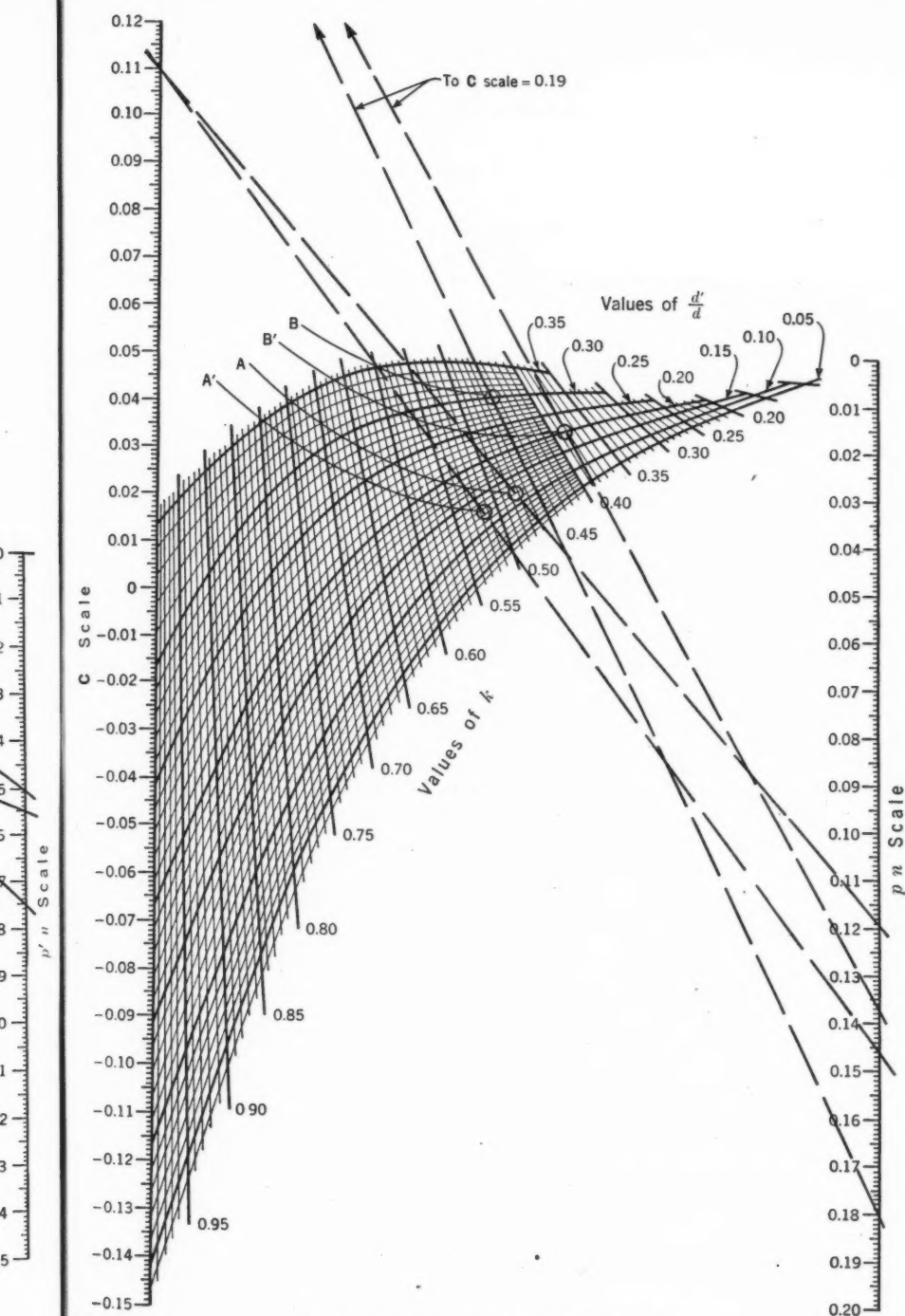


FIG. 3.—NOMOGRAM FOR  $C$ ,  $pn$ ,  $d'/d$ , AND  $k$

In designing, the procedure is to find the values of **B**, **C**, and  $d'/d$  from the given data and then to determine the values of  $k$ ,  $p' n$ , and  $p n$  from the nomograms.

**Example 1. Design of the Reinforcement for a Case of Bending and Direct Stress.**—It is required to find the longitudinal reinforcement for an eccentrically loaded column or a section of an arch ring, 12 in. wide and 18 in. deep with the reinforcement 2 in. from the surface of the concrete. The column (or the arch ring) is to carry a load or a thrust of 30,000 lb and a bending moment of 550,000 in.-lb. The maximum allowable unit stresses are 1,000 lb per sq in. in the concrete and 20,000 lb per sq in. in the steel, and  $n$  is equal to 10.

Referring to Fig. 4, let  $b = 12$  in.;  $d = 16$  in.;  $d' = 2$  in.;  $d'/d = 0.125$ ;  $M = 550,000$  in.-lb;  $N = 30,000$  lb;  $e = M/N = 18.33$  in.;  $e' = 25.33$  in.;  $f_c = 1,000$  lb per sq in.;  $n = 10$ ;  $e'/d = 1.583$ ; **B** (by Eq. 1b) = 0.247; and **C** (by Eq. 1c) = 0.111.

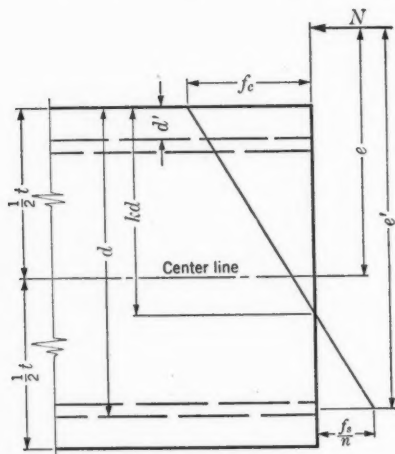


FIG. 4

In Fig. 1 lay a straight edge passing through the points **B** = 0.247 and **C** = 0.111 on the **B**-scale and the **C**-scale, respectively. Where the straight edge cuts the curve for  $d'/d = 0.125$  (interpolated), the value of  $k$  is found to be 0.470 (point A, Fig. 1). With this value of  $k$ ,  $p' n$  is found, from Fig. 2, to be 0.075; and, from Fig. 3,  $p n$  is found to be 0.118—which are the required values of  $p' n$  and  $p n$ .

On the other hand, it may be that it is desirable to run the bars through from one section to another to make the number of splices as small as practicable. In that case the given percentage of reinforcement may be used on one side and the percentage required for the

other side may be computed from the nomograms.

Suppose in this example it is desired to use 0.5% of compressive steel—that is,  $p' n = 0.050$ . Then  $k$  is found to be 0.512 (point A', Fig. 2); and, from Fig. 3,  $p n$  is found to be 0.145.

It may be noted that the sums of ( $p' n$  and  $p n$ ) in the two cases are only slightly different, but the combination obtained by using Fig. 1 is smaller. In most cases the difference will be greater than in this example.

**Example 2. Simple Bending Design with Both Tensile and Compressive Steel.**—It is required to design the reinforcement for a beam 10 in. wide and 18 in. deep with the steel 3 in. from the faces of the concrete and subjected to a bending moment of 363,000 in.-lb. The allowable unit stresses are  $f_c = 850$  lb per sq in. and  $f_s = 16,000$  lb per sq in.

In simple bending the same nomograms are used except that **B** = **C** as in Eq. 2. The given data in this example are  $b = 10$  in.;  $d' = 3$  in.;  $d = 15$  in.;

$d'/d = 0.20$ , and  $f_c = 850$  lb per sq in.; and, by Eq. 2,  $B = C = \frac{363,000}{850 \times 10 \times 15^2} = 0.190$ .

The value of  $k$  which will give the minimum value of  $(p n + p' n)$  is found to be 0.447 (point B, Fig. 1). From Fig. 2, with  $d'/d = 0.20$  and  $k = 0.447$ ,  $p' n$  is found to be zero; and, from Fig. 3,  $p n$  is found to be 0.180, or  $p = 0.018$ .

Of course, as is known,  $p n$  should be zero in simple bending when that value of  $k$  is used which gives the minimum value of  $(p n + p' n)$ .

However, 0.018 is a large percentage of tensile steel and may be considered undesirable. If so it may be reduced by using a small amount of compressive steel. In practice some of this steel is usually placed near the top of a beam for temperature or for other purposes and is frequently neglected in computing the tensile reinforcement because its effect on the tensile steel is thought to be negligible. With the nomograms in this paper it is as easy to give the compression steel its proper consideration as it is to neglect it; therefore, let it be assumed that there is 0.55% of compressive steel, or  $p' n = 0.055$ . Then  $k$  is found to be 0.385 (point B', Fig. 2); and, from Fig. 3,  $p n$  is found to be 0.137, or  $p = 0.0137$ . As should be expected, this total percentage of steel ( $0.0137 + 0.0055$ ) = 0.0192 is greater than 0.0180, the percentage of tensile steel alone.

#### EXAMPLES OF ANALYSIS

In the analysis of designs previously made, Figs. 2 and 3 are used but the procedure is different. In Fig. 2,  $p' n$  and  $d'/d$  are known but  $k$  is unknown and  $B$  is unknown because  $f_c$  is unknown. In Fig. 3,  $p n$  and  $d'/d$  are known but  $k$  is unknown and  $C$  is unknown because  $f_c$  is unknown.

However, the quantity,

$$\frac{C}{B} = \frac{\frac{e'}{d} - \left(1 - \frac{d'}{d}\right)}{\frac{e'}{d}} \dots \dots \dots (3)$$

can readily be found from the given data and is used in Figs. 2 and 3 to determine  $k$ .

*Example 3. Checking the Stresses in a Section Subjected to Bending and Direct Stress.*—It is required to check the stresses in the steel and the concrete in a member 12 in. wide and 24 in. deep subjected to a bending moment of 600,000 in.-lb and a direct thrust of 75,000 lb. The reinforcement consists of four  $\frac{3}{4}$ -in.-round bars in tension and two  $\frac{3}{4}$ -in.-round bars in compression. The embedment of the steel is 3 in. and  $n$  is equal to 10. From the given data,  $b = 12$  in.;  $d = 21$  in.;  $d' = 3$  in.;  $d'/d = 0.143$ ;  $p' n = 0.0475$ ;  $p n = 0.095$ ;  $e = M/N = 8$  in.;  $e' = 17$  in.;  $e'/d = 0.81$ ; and  $C/B$  (Eq. 3) = -0.058.

The procedure is to assume a tentative value of  $k$  and to find tentative values of  $C$  and  $B$  and of  $C/B$  from Figs. 3 and 2. The tentative value of  $C/B$  found in the first trial will probably be far from the correct value (-0.058 in this example), but the correct value can soon be reached by a few trials which can be made to approach the correct value.

The ratio  $C/B$  is always less than unity and it is observed from nomograms  $B$  and  $C$  that, when  $C/B$  is a small percentage of unity or is negative,  $k$  will be rather large. In this example,  $C/B$  is negative; therefore, with  $d'/d = 0.143$ ,  $p n = 0.095$ , and  $p' n = 0.0475$ , and knowing that  $C/B$  should be  $-0.058$ , try  $k = 0.80$ . From Figs. 3 and 2,  $k = 0.80$ ;  $C = -0.027$ ;  $B = 0.328$ ; and  $C/B = -0.082$ . The value of  $C/B$  is too low; therefore,  $k$  should be smaller. Try  $k = 0.75$  and find  $k = 0.75$ ,  $C = -0.010$ ,  $B = 0.316$ , and  $C/B = -0.032$ . This value of  $C/B$  is too high; therefore, the correct value lies between the two. Try  $k = 0.77$  and find:

Variable	$k = 0.770$	$k = 0.775$
$C$ .....	$-0.017$	$-0.019$
$B$ .....	$0.320$	$0.321$
$C/B$ .....	$-0.053$	$-0.059$

These results are as close as the accuracy of the data justifies; therefore, with  $k = 0.775$  and  $B = 0.321$ ,  $f_c$  and  $f_s$  are found from the formulas—

$$f_c = \frac{N e'}{B b d^2} \dots \dots \dots (4a)$$

and

$$f_s = n f_c \frac{1 - k}{k} \dots \dots \dots (4b)$$

to be, respectively,  $f_c = \frac{75,000 \times 17}{0.321 \times 12 \times 21^2} = 750$  and  $f_s = 10 \times 750 \frac{0.23}{0.77} = 2,240$ .

*Example 4. Checking the Stresses in a Member with Both Tensile and Compressive Reinforcement Subjected to Simple Bending.*—Given a member 12 in. wide and 15 in. deep subjected to a bending moment of 300,000 in.-lb, the reinforcement consists of two 1-in.-sq bars in tension and one 1-in.-sq bar in compression; the embedment of the steel is 3 in.; find  $f_c$  and  $f_s$ .

The given data are  $b = 12$  in.;  $d = 12$  in.;  $d' = 3$  in.;  $d'/d = 0.25$ ;  $p = 0.0139$ ; and  $p' = 0.007$ . In simple bending (Eq. 2),  $B = C = \frac{M}{f_c b d^2}$ ;  $C/B = 1$ ; and

$$f_c = \frac{M}{B b d^2} = \frac{173.6}{B} \dots \dots \dots (5)$$

As in Example 3, this problem must be solved by assuming tentative values of  $k$  until the value is found which makes  $B = C$  for the given values of  $p' n$ ,  $p n$ , and  $d'/d$ . The variable  $B$  is always positive; therefore, when  $C = B$ ,  $C$  is also positive and, when  $B$  and  $C$  are equal, the value of  $k$  is rather low. Try  $k = 0.3$ ; from Fig. 3 it is noted that  $C$  would be unreasonably large. Therefore, try  $k = 0.40$ ; from Fig. 3 find  $C = 0.182$  ( $C$ -scale produced); and from Fig. 2 find  $B = 0.194$ . Then  $C/B = 0.94$ .

Try  $k = 0.38$  and find  $C = 0.194$ ;  $B = 0.184$ ; and  $C/B = 1.05$ . It is apparent that  $k$  must be about halfway between 0.38 and 0.40 or 0.39; thus:

$k = 0.39$ ;  $C = 0.189$ ;  $B = 0.189$ ;  $C/B = 1.00$ ;  $f_c$  (Eq. 5) =  $\frac{173.6}{0.189} = 918$ ;  
and, by Eq. 4b,  $f_s = 9,180 \frac{0.61}{0.39} = 14,350$ .

It is usually not necessary to compute  $f'_s$  because its value is always less than  $n f_c$ .

From these examples it is apparent that these nomograms can be used for any problem in either bending and direct stress or in simple bending in rectangular sections of reinforced concrete, and they are believed to have considerable merit in comparison with other methods in current use.

## APPENDIX I. DERIVATION OF BASIC FORMULAS

In the derivation of the two fundamental formulas on which Figs. 2 and 3 depend, the accepted theory of reinforced concrete is adhered to without modification and the nomenclature is standard and well known.<sup>2</sup> From the condition for static equilibrium,  $\Sigma M = 0$ , taking moments about the tensile steel:

$$N e' = \frac{1}{2} f_c k b d^2 (1 - \frac{1}{2} k) + f_c p' n b d^2 \frac{k - \frac{d'}{d}}{k} \left( 1 - \frac{d'}{d} \right) \dots \dots (6)$$

which may be written

$$\frac{1}{2} k (1 - \frac{1}{2} k) + p' n \frac{k - \frac{d'}{d}}{k} \left( 1 - \frac{d'}{d} \right) = \frac{N e'}{f_c b d^2} \dots \dots \dots (7)$$

From the condition that  $\Sigma H = 0$ ,

$$N = \frac{1}{2} f_c k b d + p' n f_c b d \frac{k - \frac{d'}{d}}{k} - p n f_c b d^2 \frac{1 - k}{k} \dots \dots \dots (8)$$

which may be written

$$\frac{1}{2} k + p' n \frac{k - \frac{d'}{d}}{k} - p n \frac{1 - k}{k} = \frac{N}{f_c b d} \dots \dots \dots (9)$$

Multiply Eq. 9 by  $1 - \frac{d'}{d}$ ,

$$\frac{1}{2} k \left( 1 - \frac{d'}{d} \right) + p' n \frac{k - \frac{d'}{d}}{k} \left( 1 - \frac{d'}{d} \right) - p n \frac{1 - k}{k} \left( 1 - \frac{d'}{d} \right) = \frac{N}{f_c b d} \left( 1 - \frac{d'}{d} \right) \dots \dots \dots (10)$$

Subtract Eq. 10 from Eq. 7,

$$-\frac{1}{6} k^2 + \frac{1}{2} k \frac{d'}{d} + p n \frac{1 - k}{k} \left( 1 - \frac{d'}{d} \right) = \frac{N e'}{f_c b d^2} - \frac{N}{f_c b d} \left( 1 - \frac{d'}{d} \right) \dots (11)$$

<sup>2</sup>"Bending and Direct Stress in Reinforced Concrete," by W. P. Linton and Thomas P. Revelise, *Public Roads*, October-November-December, 1943, p. 268, Eqs. 7 and 8.

From Eq. 7,

$$p' = \frac{\frac{N e'}{f_c b d^2} - \frac{1}{2} k (1 - \frac{1}{3} k)}{n \frac{k - \frac{d'}{d}}{k} \left(1 - \frac{d'}{d}\right)} \dots \dots \dots (12)$$

From Eq. 11,

$$p = \frac{\frac{N}{f_c b d} \left[ \frac{e'}{d} - \left(1 - \frac{d'}{d}\right) \right] + \frac{1}{2} k \left( \frac{1}{3} k - \frac{d'}{d} \right)}{n \frac{1 - k}{k} \left(1 - \frac{d'}{d}\right)} \dots \dots \dots (13)$$

From Eq. 12, substituting **B** from Eq. 1b,

$$p' = \frac{k}{n \left(k - \frac{d'}{d}\right) \left(1 - \frac{d'}{d}\right)} [\mathbf{B} - \frac{1}{2} k (1 - \frac{1}{3} k)] \dots \dots \dots (14a)$$

and, from Eq. 13, substituting **C** from Eq. 1c,

$$p = \frac{k}{n (1 - k) \left(1 - \frac{d'}{d}\right)} \left[ \mathbf{C} + \frac{1}{2} k \left( \frac{1}{3} k - \frac{d'}{d} \right) \right] \dots \dots \dots (14b)$$

## APPENDIX II. CONSTRUCTION OF NOMOGRAMS

A nomogram can be constructed for any formula containing four variables  $u$ ,  $v$ ,  $r$ , and  $s$  which can be written in the form:<sup>3</sup>

$$U F_1(r, s) + V F_2(r, s) = 1 \dots \dots \dots (15)$$

in which  $U$  is a function of  $u$  alone and  $V$  is a function of  $v$  alone and  $F_1(r, s)$  and  $F_2(r, s)$  are two functions of  $r$  and  $s$ .

Eqs. 14 may be written as follows:

$$\mathbf{B} \frac{k}{\frac{1}{2} k^2 (1 - \frac{1}{3} k)} + (-p' n) \frac{\left(k - \frac{d'}{d}\right) \left(1 - \frac{d'}{d}\right)}{\frac{1}{2} k^2 (1 - \frac{1}{3} k)} = 1 \dots \dots (16a)$$

and

$$\mathbf{C} \left[ - \frac{k}{\frac{1}{2} k^2 \left( \frac{1}{3} k - \frac{d'}{d} \right)} \right] + (-p n) \frac{(1 - k) \left(1 - \frac{d'}{d}\right)}{\frac{1}{2} k^2 \left( \frac{1}{3} k - \frac{d'}{d} \right)} = 1 \dots \dots (16b)$$

<sup>3</sup>"Handbook of Mathematics for Engineers," by Edward V. Huntington, McGraw-Hill Book Co., Inc., New York, N. Y., 1st Ed., 1918.

Eqs. 16a and 16b are both in the same form as Eq. 15 and from them the two nomograms for **B** and **C** (Figs. 2 and 3) were constructed. In Eq. 16a,  $k$  and  $\frac{d'}{d}$  correspond with  $r$  and  $s$  in Eq. 15 and **B** corresponds with  $U$  and  $(-p'n)$  corresponds with  $V$ . Therefore,  $U = \mathbf{B}$  and  $V = -p'n$ , and

$$F_1(r,s) = \frac{k}{\frac{1}{2}k^2(1 - \frac{1}{2}k)} \dots \dots \dots (17a)$$

and

$$F_2(r,s) = \frac{\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)}{\frac{1}{2}k^2(1 - \frac{1}{2}k)} \dots \dots \dots (17b)$$

To construct the nomogram for **B** (Fig. 2) from Eq. 16a draw a pair of  $x, y$  axes, and through the point  $x = 1$  draw a third axis parallel to the  $y$ -axis which may be called the  $y'$ -axis. The  $x$ -axis and the  $y$ -axis need not be at right angles, but the  $x$ -axis may be sloped up at any angle desired as shown in Fig. 5.

The  $y$ -axis will ultimately become the **B**-scale and  $y'$ -axis will become the  $(p'n)$ -scale. Therefore, choose a unit for  $y$  such that the **B**-scale and the  $(p'n)$ -scale will be of the desired length.

Fix the point  $(x = 0, y = 0)$  on the  $y$ -scale and  $(x = 1, y = 0)$  on the  $y'$ -scale and connect those points by a straight line which is the  $x$ -axis. (The  $x$ -axis is not shown in the finished nomogram.) In this case, since **B** is always positive and  $(-p'n)$  is always negative, the height of the nomogram would have to be as high as the sum of the extreme values of **B** and  $p'n$  if the  $x$ -axis were made horizontal. Therefore, it is desirable to slope the  $x$ -axis up so that the lowest point on the  $y'$ -axis (the maximum value of  $p'n$ ) will be about on the same level with the lowest point of the  $y$ -axis.

Next a network of curves is drawn by plotting the points:

$$x = \frac{F_2(r,s)}{F_1(r,s) + F_2(r,s)} \dots \dots \dots (18a)$$

and

$$y = \frac{1}{F_1(r,s) + F_2(r,s)} \dots \dots \dots (18b)$$

This can always be done by giving one of the variables (say,  $r$ ) successive constant values (0, 0.1, 0.2, ..., 0.9, 1.0) in the equations, and plotting the curves for each of the constant values of  $r$ . Each curve thus plotted is marked with the appropriate value of  $r$  and each point plotted for each  $r$ -curve is marked with the appropriate value of  $s$ . Then the corresponding values of  $s$  are connected by smooth curves and each curve is marked with the appropriate value of  $s$ .

However, in many cases, it is possible to eliminate either  $r$  or  $s$  from Eqs. 18a and 18b and to obtain one equation containing only  $x, y$ , and  $r$  or  $x, y$ , and  $s$ , from which one family of curves can be plotted directly.

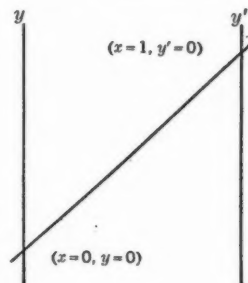


FIG. 5

In this case by substituting the values of  $F_1(r,s)$  and  $F_2(r,s)$  from Eqs. 17a and 17b in Eqs. 18a and 18b, there results

$$x = \frac{\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)}{k + \left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)} \dots\dots\dots (19a)$$

and

$$y = \frac{\frac{1}{2} k^2 \left(1 - \frac{1}{3} k\right)}{k + \left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)} \dots\dots\dots (19b)$$

The ratio  $d'/d$  may be eliminated from Eqs. 19 giving

$$y = \frac{1}{2} k \left(1 - \frac{1}{3} k\right) (1 - x) \dots\dots\dots (19c)$$

from which the  $k$ -curves may be easily plotted. It is apparent from Eq. 19c that the  $k$ -curves are all straight lines passing through the point  $(x = 1, y = 0)$ ; and, when  $x = 0, y = \frac{1}{2} k \left(1 - \frac{1}{3} k\right)$ .

The  $(d'/d)$ -curves can then be plotted by giving  $d'/d$  successive constant values in Eq. 19b.

Next the **B**-scale is constructed by plotting the values of **B** along the  $y$ -axis and the  $(p'n)$ -scale is constructed by plotting the values of  $p'n$  along the  $-y'$ -axis. The  $x$ -axis is then erased and the nomogram **B** (Fig. 2) for the variables **B**,  $p'n$ ,  $k$ , and  $d'/d$  in Eq. 16a is completed. When any three of the four variables are known, the fourth can be found immediately.

The nomogram **C** (Fig. 3) for the four variables **C**,  $p'n$ ,  $k$ , and  $d'/d$  in Eq. 16b is constructed similarly. In this case,

$$F_1 = \frac{-k}{\frac{1}{2} k^2 \left(\frac{1}{3} k - \frac{d'}{d}\right)} \dots\dots\dots (20a)$$

$$F_2 = \frac{-(1-k)\left(1 - \frac{d'}{d}\right)}{\frac{1}{2} k^2 \left(\frac{1}{3} k - \frac{d'}{d}\right)} \dots\dots\dots (20b)$$

$$x = \frac{(1-k)\left(1 - \frac{d'}{d}\right)}{k + (1-k)\left(1 - \frac{d'}{d}\right)} \dots\dots\dots (20c)$$

and

$$y = - \frac{\frac{1}{2} k^2 \left(\frac{1}{3} k - \frac{d'}{d}\right)}{k + (1-k)\left(1 - \frac{d'}{d}\right)} \dots\dots\dots (20d)$$

Eliminating  $d'/d$  from Eqs. 20c and 20d,

$$y = \frac{1}{2} k (1 - \frac{1}{2} k) - \frac{1}{2} k \left[ (1 - \frac{1}{2} k) + \frac{k}{1 - k} \right] x \dots \dots \dots (21)$$

The  $k$ -curves plotted from Eq. 21 are straight lines. The  $(d'/d)$ -curves are plotted from Eq. 20d.

The two nomograms for **B** and **C** can be used without the nomogram in Fig. 1, but it will be noted that Fig. 1 is very useful in finding the most economical values of  $k$ .

In designing the reinforcement only two of the four variables in each nomogram are known—that is, in Fig. 2, **B** and  $d'/d$  are known whereas  $k$  and  $p'n$  are unknown; and, in Fig. 3, **C** and  $d'/d$  are known whereas  $k$  and  $p'n$  are unknown.

In some cases the value of  $p'n$  or the value of  $p'n$  may be fixed by other considerations, such as by the use of the same bars as those in another section. In that case the steel in one face is fixed; then the value of  $k$  can be found from the known percentage of steel and that value of  $k$  used in finding the unknown percentage.

If neither  $p'n$  nor  $p'n$  is known, either one of them may be assumed and the other may be found after determining  $k$  from the assumed percentage; but this procedure will require several assumptions and trials before the most economical or satisfactory combination is obtained.

To obviate the assumptions and trials, Fig. 1 was devised to give the value of  $k$  that will make the sum of  $p'n$  and  $p'n$  a minimum. To do this, Eqs. 14 were added together, giving

$$\begin{aligned} & \left( 1 - \frac{d'}{d} \right) (p'n + p'n) \\ &= \frac{\frac{2}{3} \left( 1 - \frac{d'}{d} \right) k^3 - \left\{ B - C + \frac{1}{2} \left[ 1 - \left( \frac{d'}{d} \right)^2 \right] \right\} k^2 + \left( B - C \frac{d'}{d} \right) k}{- \left[ k^2 - \left( 1 + \frac{d'}{d} \right) k + \frac{d'}{d} \right]} \dots \dots \dots (22) \end{aligned}$$

Eq. 22 was differentiated with respect to  $k$  and the derivative placed equal to zero, from which the equation—

$$\begin{aligned} - \frac{d'}{d} (1 - k)^2 B + \left( k - \frac{d'}{d} \right)^2 C &= \frac{2}{3} \left( 1 - \frac{d'}{d} \right) k \left\{ k^3 - 2 \left( 1 + \frac{d'}{d} \right) k^2 \right. \\ &+ \left. \frac{3}{4} \left[ 1 + 6 \frac{d'}{d} + \left( \frac{d'}{d} \right)^2 \right] k - \frac{3}{2} \frac{d'}{d} \left( 1 + \frac{d'}{d} \right) \right\} \dots \dots \dots (23) \end{aligned}$$

—was obtained and was written

$$\frac{(-B) \frac{d'}{d} (1 - k)^2 + C \left( k - \frac{d'}{d} \right)^2}{\delta} = 1 \dots \dots \dots (24a)$$

in which, to simplify topography,

$$\delta = \frac{2k}{3} \left( 1 - \frac{d'}{d} \right) \left\{ k^3 - 2k^2 \left( 1 + \frac{d'}{d} \right) + \frac{3k}{4} \left[ 1 + 6 \frac{d'}{d} + \left( \frac{d'}{d} \right)^2 \right] - \frac{3d'}{2d} \left( 1 + \frac{d'}{d} \right) \right\} \dots \dots \dots (24b)$$

which is the same form as Eq. 15. Eqs. 24 also represent the form desired for the construction of Fig. 1 which was drawn in a manner similar to that used for Figs. 2 and 3, although in this case it is impractical to eliminate either  $k$  or  $d'/d$  from the equations for  $x$  and  $y$  corresponding to Eqs. 20c and 20d.

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# AMERICAN SOCIETY OF CIVIL ENGINEERS

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## DISCUSSIONS

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### PANAMA CANAL—THE SEA-LEVEL PROJECT A SYMPOSIUM

#### Discussion

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BY HARRY O. COLE, OLE P. ERICKSON, CLARENCE S. JARVIS,  
ANSON MARSTON, AND RALPH Z. KIRKPATRICK

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HARRY O. COLE,<sup>117</sup> M. ASCE.—From the nature of the papers in this Symposium, together with another extremely able paper,<sup>118</sup> it is clear that the investigation of the canal under Public Law No. 280 (Seventy-ninth Congress, First Session) was solely directed toward securing the authorization of the predetermined objective of a sea-level project on the premise that the present lock canal "is totally lacking in security to meet the future needs of national defense," and that the only alternative is a sea-level canal.

It appears that the investigation reported in the Symposium was not directed toward securing any major improvement in the operating deficiencies of the present lock canal. At least if there was any such objective, the writer has not found it in the Symposium papers.

The first major contribution toward the solution of operating difficulties in the canal was developed by Miles P. DuVal<sup>119</sup> in a paper published in 1947. The plan presented in that paper—its recording and analysis of accidents, hazards, and difficulties encountered by ships in transiting the canal, over a period of 30 years—was so revealing and informative that it has apparently affected all subsequent engineering thinking at Panama. As all informed engineers with experience in building the Panama Canal know, the problems there are not problems of complexity as much as they are of magnitude.

Viewing with alarm the breakneck speed at which the United States has been spending hundreds of billions of dollars during the 7 years since 1941

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NOTE.—This Symposium was published in April, 1948, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: June, 1948, by Hans Kramer, and Philip G. Nichols; September, 1948, by George B. Pillsbury, Kenneth S. M. Davidson, Joel D. Justin, W. H. McAlpine, W. E. R. Covell, William Herbert Hobbs, Hibbert Hill, Kenneth C. Reynolds, Gregory P. Tschebotarioff, Charles W. Dohn, and Donald F. Horton; October, 1948, by E. Montford Fucik, Charles M. Romanowitz, and Raphael G. Kazmann; November, 1948, by George B. Massey, William Allan, and Boris A. Bakhmeteff; and December, 1948, by Robert C. Sheldon, F. W. Edwards, and H. R. Cedergren.

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<sup>118</sup> *Congressional Record*, April 22, 1948, p. A2565.

<sup>119</sup> "The Marine Operating Problems, Panama Canal, and the Solution," by Miles P. DuVal, *Proceedings*, ASCE, February, 1947, p. 161.

(and still doing so) it is clear to all thinking men that unless the nation safeguards its resources, present investments, and future expenditures, it will further seriously jeopardize the national security at a critical time.

It is fair to ask the question, and insist upon a truthful and an unbiased answer: Is an expenditure of \$2,500,000,000 justified at Panama or elsewhere on the Isthmus for any canal, lock or sea level, under any circumstances?

The recognized superiority of the Panama Canal as a navigable waterway for ocean vessels over any feasible canal at Nicaragua, Tehuantepec, the Atrato River, or elsewhere, and the high vulnerability of any type of canal to the atomic bomb certainly makes the very idea of expending \$2,500,000,000 (and probably much more) for a sea-level project at Panama, or for a canal at any other location, entirely indefensible.

The national resources of the United States should not be squandered in a futile Isthmian project at Panama, or any other place, that would inevitably divert funds and resources from other urgently needed projects and defense programs in the United States.

From evidence presented elsewhere by the writer,<sup>120,121</sup> it is very clear that the great fundamental defect in the original design of the Panama Canal was the separation of the Pacific locks into two groups and the placing of one of them squarely across the south end of the Culebra Cut (later Gaillard Cut) at Pedro Miguel. The writer has given the definite reasons why that was done, and has proved beyond reasonable doubt that, had not that decision been reached and carried out at that time, the United States would have had no canal at Panama during World War II!

The Third Locks Project of 1939, had it been completed (it was abandoned in 1942), would not have been the cure, but it would have greatly aggravated and increased the difficulties at Pedro Miguel because "this project was not navigationally sound!

Also, revived with the 1939 Third Locks Project was the idea of "converting" the Panama Canal to sea level, with a plan that had not been studied adequately. The Panama sea-level project reported in the Symposium apparently grew out of that plan and will make it necessary to abandon a great part of the present canal. It is not a true "conversion" but really a great project for a new Panama Canal.

Authorities who are considered well informed have expressed the opinion publicly that any type of canal is definitely vulnerable to the atomic bomb and that the only function of the canal is to transport ships safely and conveniently. It is as self-evident now, as it was when the Panama Canal was constructed, that a canal formed by large lakes affords safer and more convenient navigation than any canal that has a restricted, canyon-walled, channel 30 miles long and whose tidal waters are changing in velocity and direction throughout every day in the year.

Furthermore, the proposed huge "tidal lock control works" with a lock 200 ft by 1,500 ft, contemplate an independent 750-ft wide, "navigable-pass channel" controlled by a 6,000-ton, 6-link, chain-gate system. This project

<sup>120</sup> *Congressional Record*, May 4, 1948, p. A2872.

<sup>121</sup> *Ibid.*, May 21, 1948, p. A3338.

definitely rules out the possibility of "converting" the lock canal to a true sea-level canal.

According to one of the highest naval authorities, the expenditure of building a so-called sea-level canal, or even huge locks, to expedite the transfer of 2 or 3 "super-width" Navy ships from one ocean to the other, is not justified, since the United States now has a two-ocean Navy, and since these "super-width" ships can readily go around Cape Horn when it becomes necessary for them to be transferred from one ocean to the other.

One other technical phase of this subject is of the most vital importance to the American people. The estimated first cost for "converting" the lock canal to sea level, of \$2,483,000,000 alone (which appears to be grossly underestimated), is prohibitive, unless the cost is to be borne by the national defense budget. The annual interest of 3%, not to mention paying off the debt, amounts to about \$75,000,000. This would mean an increase in toll charges of about \$2 per ton over and above the present toll charge of \$1. Even now, shipping interests are clamoring for a reduction in toll charges. If authorized as a defense project with a debt to be paid off in the next 100 years, the American taxpayers would have to do it.

A review of the facts and references as set forth in the foregoing should give a better perspective of the canal investigation that is under discussion.

Apparently the major premise (national defense), on which the recommendation for "converting" the canal to sea level was based, has now definitely been removed.

The sea-level plan recommended in the Symposium proposes to abandon the present high lake type of canal (which has demonstrated its convenient operation since 1914) and substitute a sea-level canal—at an expenditure of untold billions of dollars with no positive proof that in the end it will prove to be as good, or any better, or more economical, or safer for transiting ships, than an improved lake-type canal at only a fraction of the cost and the time to build!

OLE P. ERICKSON,<sup>122</sup> M. ASCE.—In the ninth Symposium paper there were discussed various methods of excavation, such as large shovels, dredges of the dipper and bucket-ladder types with dump scow disposal, and 40-in. hydraulic dredges.

The proposed hydraulic dredges of only 40-in. discharge seem to the writer to be too small. Consideration should be given to much larger dredges, say, with 60-in. discharge pipe lines. It is fully realized that such dredges with a theoretical capacity four times that of the 30-in. dredges now in use, and two and a fourth times the capacity of the proposed 40-in. dredges, are unusual. However, this is a project where large and special equipment with high production capacity can be used advantageously. The friction loss in the 60-in. discharge line would be about half that in the 40-in. discharge line, which would mean a great saving in horsepower. Larger solids could pass through the pump and pipe line, thus reducing delays from this source.

<sup>122</sup> Cons. Engr., Bureau of Reclamation, Boulder City, Nev.; Pres., Erickson Eng. Co., Cons. Engrs., Tampa, Fla.

There is no particular reason why 60-in., or larger, discharge pipe line dredges, with one or two booster plants, cannot be designed and built to pump the excavated material directly into the dam or levee sections in Gatun Lake. Furthermore, the dams or levees could be pumped to the final height required and could of course be made of any cross sections desired. By allocation of the materials available for fill there should be no need for pipe lines more than 20 miles long. The average pipe line lengths would apparently be 15 miles or less.

With variable cutter speeds and required horsepower on the cutter, a 60-in. hydraulic dredge could excavate all the softer rocks and shales without blasting, except for bringing down high banks. Proper cutter design would screen out all rock or materials that would not pass through the dredge pump. The cutter would have a variable rotative speed up to 1,500 ft or more per min, and the cutter teeth would split the rock by impact rather than with pressure as do the dipper type and bucket-ladder type dredges with their slow bucket and tooth speeds.

In the rainy season hydraulic dredges would operate without delays from rains, which would slow down other methods of operation, because no shore line handling would be required, except a small amount for topping the dam sections. The sticky clay shales, which would be very nasty to handle by other methods, would make ideal hydraulic dredge pumping material.

Two of these 60-in. dredges, it is estimated, would handle approximately 70,000,000 cu yd per year if pumped into Gatun Lake or into alternate spoil areas where shore line handling would not be required. A comparison of the cost per cubic yard of unclassified material delivered by 60-in. hydraulic dredges into the dam sections and special areas in Gatun Lake with the cost by the other proposed excavating equipment, and loading into dump scows, towing, and disposing in the same spoils areas, would be very instructive.

As regards the various other types of excavating equipment contemplated, the dipper dredge can excavate from 85 ft to 90 ft and has the distinct advantage of being able to handle large boulders or rock sections. Delays may be expected, however, from possible spud and dipper stick failures in the deep dredging. The bucket-ladder type dredge cannot efficiently handle large boulders or hard rock, and it is also low in production although excellent for tin and placer mining. The dredge *Corozal*, built for work in the present Panama Canal, did not show too much promise in this type of dredging. The large shovels and draglines, 25 cu yd or larger, are ideal tools for dry excavation. However, loading into dump scows from canals and towing to Gatun Lake will create some problems. Dump scows of capacities from 3,000 cu yd to 5,000 cu yd should be given consideration for this method of disposal, instead of the 2,000-cu-yd size contemplated.

CLARENCE S. JARVIS,<sup>123</sup> M. ASCE.—The papers in this Symposium, together with the extensive underlying investigations and critical studies, have established beyond reasonable challenge the vital need for the project as clearly outlined by specialists in line of their official duty. Even the most costly and

<sup>123</sup> Cons. Engr., Salt Lake City, Utah.

apparently fantastic of the alternate routes, plans, and suggested procedures has served a good purpose in focusing attention on the more meritorious ones for final studies and revisions.

The writer is in complete agreement with those who believe that tidal regulation is unnecessary and therefore inadvisable, even though the technical studies were in order. Recalling the situation in 1933, when a tidal lock for the Cape Cod Canal in Massachusetts not only was an approved feature of the enlargement program then under way, but also was on the verge of being let to contract or other definite construction commitment until challenged by the division engineer with technical data assembled by the writer in line of duty, it is pleasing to note the years of satisfactory service without tidal control. As forecast at that time, an enlarged sea-level canal would be effective in reducing the differentials of bay surface elevations as compared with those previously observed near the canal terminals.

If the maximum differentials of about 7 ft fail to produce objectionable velocities in the 9-mile length of the Cape Cod Canal between approach channels, then it appears doubtful that a 10-ft or 11-ft head would produce flow velocities much greater, if as great, at the Panama Canal terminals, nearly four times as far apart, even though the canal cross section will be nearly double in area.

The proposed tidal lock for the Cape Cod Canal was regarded by the writer as a nearly useless obstruction, entailing further hazard to shipping when vessels slowed down and maneuvered for anchorage.

Although recognizing the apparently overwhelming weight of authoritative opinion already in favor of the Panama sea-level conversion route either along or near the present alinement, the writer has as yet been unable to reconcile the following elements and factors that should be considered:

1. Would not the Panama parallel sea-level canal as proposed fully justify the additional estimated cost of \$800,000,000, or approximately one half of the total construction and operating costs of the present installation, together with accrued interest at 3% per annum until 1960? Advantages would include assurance of noninterference with canal traffic during construction, a dependable stand-by or alternate route, conservation of the hydroelectric power and flood-control facilities and potentialities, and avoidance of wet excavation totaling nearly 300,000,000 cu yd, nearly half of which would be classified as rock. Finally, some 132,000,000 cu yd of this total excavation on the present canal would be taken from depths ranging from 85 ft to 145 ft below maximum water surface, probably involving unprecedented trouble.

2. Would not the unavoidable interference with normal canal traffic during the period of construction for the Panama sea-level conversion route, together with the estimated 7-day interruption during the final stages of conversion and demolition (or possibly a much longer interruption due to unforeseen conditions), offset a considerable part of the estimated saving in construction cost? Much uncertainty attends the deep blasting and dredging of rock since small-bucket dredging for gold from alluvial bars to depths of 128 ft offers no real precedent.

3. Construction on the Panama parallel sea-level route, or such revised route as further study may dictate, would have all the advantages that come with mass production or mass excavation, with the possibility, if not the probability, of reducing unit costs below those first estimated.

4. The sea-level conversion project, substantially utilizing the present route, but grappling with massive rock at such unprecedented depths and denied the observational and other advantages that attend excavation in the dry, might well overrun the estimated costs as well as the estimated time limits; and, when it reached completion, there would be just one usable Panama route with no alternate or stand-by.

The writer has doubtless overlooked many of the basic data that led to authoritative support of the Panama sea-level conversion project in preference to all others yet suggested. However, can these in the aggregate outweigh the considerations herein listed as needing reconciliation? The writer is reassured that continuous research and review may be maintained, not only pending final decision but also throughout the construction period, because the project is in safe hands, guided by the highest concepts and ideals.

ANSON MARSTON,<sup>124</sup> PAST-PRESIDENT AND HON. M. ASCE.—This admirable Symposium on one of the greatest and most important projects in the history of engineering is of special interest to the writer. Every rereading and restudy of the nine papers, keeping in mind his own study (1929 to 1931) of the same problems as a member of the Interoceanic Canal Board, has confirmed the conclusion on first reading—that, starting immediately, the present canal should be converted into a sea-level canal.

The present canal was made a lock canal because of the eagerness of the United States to be able to send its ships across the Isthmus at the earliest possible date. Soon after the Spanish-American War, J. P. Dolliver, then United States Senator from Iowa, speaking to an Iowa audience, stated: "As for the Panama Canal, because the *Oregon* had to go around Cape Horn, you may consider it as dug"—and it was, by the earliest date possible.

The present canal has served its purpose well, in spite of some mistakes in its original planning and construction. The mistakes nearest to being inexcusable were the location of the Pedro Miguel Lock close to the Pacific end of the Gaillard Cut and the overruling of the advice of the late Gen. W. L. Sibert, M. ASCE, later to abandon work already done on it and to construct instead three-step locks at Miraflores. It is debatable whether this mistake was excusable, but the writer thinks that making the locks too small for future ship sizes was excusable; it was impracticable to predict correctly the future sizes of the large ships of the twentieth century. In 1931, locks 125 ft by 1,200 ft in size, instead of 110 ft by 1,000 ft, were adopted on advice of the Interoceanic Canal Board.<sup>6</sup> In 1939, Congress ordered these locks constructed 140 ft by 1,200 ft; and, by 1948, 200-ft by 1,500-ft locks were recommended by the United States Navy.

<sup>124</sup> Dean Emeritus of Eng., Iowa State College, Ames, Iowa.

<sup>6</sup> "U. S. Army Interoceanic Canal Board Report on Panamal Canal and Nicaragua Canal (Sultan Report), 1931." *House Document No. 139*, 72d Cong., 1st Session, 1932.

The fact is that the sizes of the largest commercial ships and warships needing to pass through the canal within the next 50 years cannot even yet be estimated with certainty. A sea-level canal with a "navigable pass" of the full dimensions of its channel is the only sure solution of this problem.

The writer is convinced that the conclusion stated in the Symposium that only a sea-level canal can insure security to meet the needs of national defense against modern air attack is absolutely correct. Consequently, it is imperative that the sea-level plan be adopted immediately and that construction be begun. The world situation is extremely dangerous. Experience in World War II with the Third Locks Project shows how hopeless it was, and would be again, to wait until another war becomes certain before starting construction on any major Panama Canal security or other improvement project.

In 1931, the late Sydney B. Williamson, M. ASCE, a member of the inter-oceanic Canal Board,<sup>6</sup> made an estimate of the probable future canal traffic across the Isthmus, on three bases, namely—(1) percentage of world fleet tonnage, past and probable future; (2) percentage of Suez Canal tonnage, past and probable future; and (3) data of average gain in actual Panama traffic, 1915 to 1930. His resulting estimates are given in Table 35.

TABLE 35.—1931 ESTIMATES OF PROBABLE PANAMA CANAL TRAFFIC

Year	Ships	Net Tons	Year	Ships	Net Tons
1940	8,600	40,300,000	1960	13,900	65,400,000
1950	11,100	52,100,000	1970	16,700	80,100,000

Comparison of Table 35 with Figs. 1 and 2 shows that the 1931 estimates for both net tons and transits are larger than those of the Symposium—which, being based on later data, may be presumed to be more accurate. Fig. 1 shows the great effect of World War I. If there is a World War III, what will be its effect? Mr. Williamson and the late Harry Burgess, M. ASCE, then Governor of the Canal Zone, estimated that the maximum capacity of the twin-locks Panama Canal would be reached from about 1960 to 1970; the symposium states 1960.

In 1931, a Nicaragua lock canal was entirely feasible, at a cost estimated at \$722,000,000 (which, of course, would be much greater for 1948). Although, its length would have been more than three times that of the Panama Canal, it was estimated that about two thirds of the traffic then crossing the American Isthmus, especially intercoastal United States shipping, would save distance by going through a Nicaraguan rival to the Panama Canal. To avert possible disastrous rivalry, either financial or international, in 1916 the United States concluded a convention with Nicaragua whereby it acquired:

"\* \* \* in perpetuity \* \* \* forever free from all taxation or other public charge, the exclusive proprietary rights necessary and convenient for the construction, operation, and maintenance of an interoceanic canal by way of the San Juan River and the great lake of Nicaragua, or by way of any other route over Nicaraguan territory \* \* \*"

In return the United States paid Nicaragua \$3,000,000 in gold.

Costa Rica, Salvador, and Honduras objected. The United States and Nicaragua held firm, but before 1931 the United States State Department gave "every assurance" that prior to the building of any canal it would enter into negotiations with Costa Rica, Salvador, and Honduras.<sup>6</sup>

In 1931, Governor Burgess, of the Panama Canal Zone, made the first report on a definite, detailed project for doing what had long been talked and written about—conversion of the Panama lock canal into a sea-level canal. For this project he gave full credit to Mr. Williamson, of the InterOceanic Canal Board, who proposed to accomplish the conversion by lowering the canal in three stages: From El. 85 to El. 53; from El. 53 to El. 27; and from El. 27 to El. 0. The final result was to be a canal 42 ft deep and 500 ft wide at the bottom, with triple-tidal locks at Miraflores, each 140 ft by 1,500 ft. The total cost was estimated to be \$1,000,000,000. It was admitted that lowering the locks by stages would involve serious hazards, admissible only after constructing third locks.

Although approving this plan then, the writer has always doubted the necessity for the tidal locks, and now is glad to learn that actual computations, using the theory of Gen. George B. Pillsbury, M. ASCE, and measurements from an undistorted generous scale model, combine to prove that the canal will be navigable through the proposed navigable pass even if the tidal locks should be destroyed.

The writer is not a dredging expert, but has studied with some care the plans for special hydraulic dredges working to a depth of 145 ft, kindly sent him in advance, with Colonel Stratton's consent, by Panama Contractors. Single-stage lowering to sea level will do away with at least most of the lock lowering hazards unavoidable with multiple-stage lowering.

The writer is much impressed by the paper by Messrs. Binger and Thompson. Great slides have been a continuing dangerous and expensive characteristic of the construction and maintenance of the present canal. To an engineer who has been interested, and to some extent engaged, in engineering research for 57 years, it is pleasing to note that the Panama Canal has served as a great research laboratory to solve the important problem of safe slopes for a sea-level Panama Canal.

It is pleasing, also, to read in the other Symposium papers the reports of the researches made on the design of the channel to afford safety in ship handling. Enlisting the participation of Panama Canal pilot captains in these researches was especially wise.

In conclusion, the writer earnestly believes that Congress should at once authorize a sea-level Panama Canal and provide funds for starting the work. The present world situation is such that it is dangerous to procrastinate. The expenditure of from \$250,000,000 to \$300,000,000 annually for 10 years is small in comparison with the billions of dollars being spent to prevent another war, and the hundreds of billions of dollars another war would cost. A sea-level canal, safe to transit the largest war and commercial ships at any time, is vital to the national security. It is unthinkable that the United States should ever again be forced to send them around Cape Horn, like the *Oregon*, in 1898.

RALPH Z. KIRKPATRICK.<sup>125</sup>—An effective answer to the basic thesis of this Symposium is a paper published in 1947.<sup>119</sup> To the writer, that paper is an infrared light through the "confusion of ideas" begot by the various plans for the best method of modernizing and enlarging this tropical waterway. The term "confusion of ideas" was used by the late John F. Stevens, Hon. M. and Past-President, ASCE, the chief engineer of the Panama Canal during the earlier part of the construction period. Although he used it as early as 1906, oddly his subject was almost identical to the current one.

Perhaps the sufficiency of the water supply in the driest dry season may be questioned, in what is termed either the "Balanced Lake Plan" or the "Terminal Lake Plan," because it is a fact that the balance which the Madden Lake Dam made possible in flood handling, power making, and water supply for navigation has been slightly upset by the subsequent additional power units operating at the Madden and Gatun hydroelectric plants. Also, the proposed wider locks will use more water.

Fortunately this problem is easily solved: During the reconstruction period (without interference to navigation) simply dredge the bottom of Gaillard Cut from El. +40 to El. +37½, the ruling bottom elevation of Gatun Locks (that is, the tops of the emergency dam sills); and, since the Terminal Lake cancels the Gaillard Cut lockage surges, there is no further need for the allotment of an extra 1½ ft of dry-season prism-storage water. Thus, a realistically useful storage water prism of 2.67 + 1.5, or 4.2 ft atop Gatun Lake becomes available for dry season use. In terms of water, this is about 18,600,000 cu ft, or an increase of nearly 75%.

Incidentally, during the fiscal years 1940-1945, the annual power generation in the Canal Zone increased from about 100,000,000 kw-hr to 266,800,000 kw-hr; and, subsequently, the decrease has been only slight. The four turbines at the Gatun plant are now (1948) augmented by two new and larger ones. Under the sea-level plan, that plant must close, and a virtually new electrical system for the Canal Zone must be designed and built, with but minor salvage. All that useful power-making water from Gatun Lake will pass to the sea through the proposed East and West Diversion systems! Compared to the present system or proposed Terminal Lake system the contrast in efficiency and dollar savings will be marked!

The capacity of Gatun Lake to absorb safely, and to make profitable use of the 60 in. to 150 in. of annual rainfall (8 months) in the Chagres River Valley has been tested for one third of a century without a failure. It is one of the triumphs of the pioneers of the Panama Canal that their potential worst enemy has actually been made a best asset. Why gamble? Sooner or later "bugs" are nearly certain to bedevil the operations of a sea-level canal through faults in the proposed extensive and complicated West and East Diversion systems. What of those miles of levees, dams, and spillways proposed on foundation conditions still virtually unknown?

The slides in the 8 miles of Gaillard Cut have a soul-searing history; but those troubles were caused by geological, and maybe other, conditions above

<sup>125</sup> Rochester, N. Y.; formerly Chf. of Surveys-Chf. Hydrographer, Panama Canal, Canal Zone.

the present canal bottom elevation of +40. On the face of it, going down 100 ft further to El. -60 (see Figs. 98 and 99) is daringly unprecedented even to the point of recklessness. Furthermore what is known of the probable bank and bottom stabilities of that other 20 miles of proposed canal now safely hidden under Gatun Lake? Nothing is known, except geological conjectures!

What foundation tests have ever determined that sufficient bearing values exist for those proposed dams across the Peña Blanca and Caño Quebrado swamps? Those deep quagmires of Atlantic mucks have been forgotten under Gatun Lake for 37 years! Nevertheless, there those important dams are to be found, on the maps of the proposed sea-level canal. For a long time, those swamps were the dreads and troubles of the pioneer French and American engineers—now sanguine hopes make them dam foundations!

Apparently through well-meaning but oversanguine proposals, the future safe and smooth transit of world commerce is being endangered. It must not be!

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# AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

## DISCUSSIONS

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### THE LOGICAL SYSTEM OF FREIGHT RATES

#### Discussion

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BY RAPHAEL H. COURLAND

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RAPHAEL H. COURLAND,<sup>2</sup> ASSOC. M. ASCE.—“The Logical System of Freight Rates” and the implied definition of “fair” rates are reasonable as far as they go, but they do not take into consideration certain pertinent factors that bear no relation to an engineering-economic analysis of transportation, as such. These factors, in some instances, are the only determining elements in rate setting; in fact, sometimes they constitute the sole justification for the existence of the carrier. Thus, it is not always true that “The object in setting these ratios is to cause traffic to move along the most economical lines \* \* \* .”

Immediate military expediency, flood control, hydroelectric power development, long-range national research and development of certain natural resources, national policy in support of the exploitation of external natural resources, encouragement of “infant” or young industries and travel facilities (in furtherance of national military potential), use of mail service subsidy—all these have been, are, and will continue to be, factors of major influence in the national transportation system. Because of them, the generalization—“only if carriers meet all their own expenses out of their own receipts will the logical system of freight rates work smoothly”—cannot be considered absolute.

Hypothetically, if a commercial enterprise such as a carrier is to be considered self-sufficient, rate setting must be such that income will amortize the capital investment, pay operating expenses and maintenance, permit normal replacement, and yield attractive profits. When the enterprise is not self-sufficient, and when the operation of the carrier is maintained only by continuous subsidy, then it is evident that some purpose besides that of commercial transportation is being served. Assuming that any one, or any combination of the noncommercial purposes listed in the second paragraph of this discussion is being served by the carrier, then the problem is that of paying for the noncommercial purpose being served. Such a purpose normally has its inception with some agency (federal, local, or a combination of both) which sees to its implementation, and sometimes, its subsidy.

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NOTE.—This paper by Charles L. Hall was published in November, 1948, *Proceedings*.

<sup>2</sup> Civ. Engr., Maurice Courland and Son, New York, N. Y.

Shall the freight rate on the carrier that is not self-sufficient be such that it would be the "fair" rate on a hypothetical self-sufficient carrier rendering the same commercial service on a competitive basis in a well-developed area, and thus permit mail service subsidy or direct subsidy to satisfy the resulting annual deficit? Or, shall those who use the carrier be required to pay a rate that will make the enterprise solvent on the basis of commercial use only?

The answer to the question seems to lie in the definition of "fair." If the carrier serves a commercial purpose only, and is operated as a monopoly or quasi-monopoly, the concept of "fair" rates as defined by Colonel Hall would appear to be valid. However, if the carrier serves, in part, a noncommercial purpose, rates should be determined by the character of that purpose.

If the noncommercial purpose bears no relation to commerce (for example, military expediency), the agency initiating the action to implement that purpose should be obliged to stand its cost through direct subsidy or service subsidy; and rates should be a matter of maintenance and operating costs only (no amortization or profit included).

If the noncommercial purpose is of a long-range commercial venture (such as shifting or encouragement of industry, or travel facilities), the rate should be one that could initially compete with similar existing services; and annual deficits must be satisfied by the initiating agency. In these cases it is assumed that within a reasonable period the development of the areas involved would be adequate to carry the carrier without subsidy. However, it must be noted in this regard that some industries, transportation facilities, and settlements are long-range ventures into military potential. In such cases, as long as the need for the potential is judged to exist, rates should be set so as to attract personnel and equipment that would accomplish the ultimate military objective, with no regard to an engineering-economic analysis. Deficits, of course, should then be met indefinitely by subsidy.

If the nonmilitary purpose is related indirectly to the economic well-being of a region, another factor must be considered. Many of these enterprises, especially related to water, are of dual and sometimes triple purpose. Water transportation is frequently the concomitant benefit of the basic enterprise when flood control and hydroelectric power development alone are considered. At times, river canalization is the primary objective; but the indisputable fact remains that without too basic a modification of design concepts, and with a proper sequence of storage and release, all three benefits may be secured. In general, it is assumed that the rise in land values resulting from flood protection, added power, and reliable transportation could amortize an entire project within a 50-year period. In such cases it would seem reasonable that the increased revenue in the region derived from land taxes, new industries, and commerce should be applied toward amortization of the capital investment, whereas maintenance and operating costs should be satisfied by electric rates (for power) and by lock and canal rates (for river transportation).

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## DISCUSSIONS

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### REVIEW OF SLOPE PROTECTION METHODS REPORT OF THE SUBCOMMITTEE ON SLOPE PROTECTION OF THE COMMITTEE ON EARTH DAMS OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION

#### Discussion

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BY WALTER F. EMMONS AND ADOLF A. MEYER

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• WALTER F. EMMONS,<sup>41</sup> ASSOC. M. ASCE, AND ADOLF A. MEYER,<sup>42</sup> M. ASCE.—In many instances the permanency of embankment slopes is an important condition for the safety of structures. Great expenditures have been made and are being made to obtain safe slopes; yet most of this work is conceived with hardly any relation to the fundamental forces involved, and it lacks completely the preciseness usually associated with engineering design. The situation compares closely with the design of earth structures as of about 1928. In its report the subcommittee clearly shows the meagerness of available design data for slope protection. It is hoped that its review will help to instigate the hydraulic investigation so obviously needed. A carefully planned program of model testing could supply the essential basic data from which a dependable design method could be developed.

Against such an optimistic attitude, it may well be pointed out that "dumped stone riprap," the primary construction material in question, does not lend itself to any sharp classification in size and shape. Any hydraulic investigation would have to assume such a classification as feasible. Under these circumstances it is well to point to the experiences in soil mechanics. Whenever a novel treatment of the soil was shown to offer positive advantages, the construction industry did not fail to provide a practical method to implement it.

For any embankment slope facing a body of water, the protective work starts with the meteorological evidence. The knowledge of frequency, intensity, and direction of storms is likely to be limited for most locations. The situation when selecting "the design storm" is similar to the adoption of a

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NOTE.—This report was published in June, 1948, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: October, 1948, by Howard J. Hansen, William P. Creager, Henry H. Jewell, Joe W. Johnson, and Martin A. Mason; and December, 1948, by R. M. McCrone, and Harold Weggel.

<sup>41</sup> Asst. Chf., Civ. Design Div., TVA, Knoxville, Tenn.

<sup>42</sup> Chf., Civ. Design Div., TVA, Knoxville, Tenn.

"design flood." It is a problem of probability in which storms of record at the site, storms of record in meteorologically related territory, and the magnitude of damage which may be involved, have to be evaluated.

From there on the problem is one of physical relations throughout. Wave characteristics can obviously be established in relation to storms and the dimensions of a body of water. A certain type of wave has a fixed amount of energy which is to be dissipated against riprap. Depending on slope and roughness of the riprap, the dissipation forces will vary.

The transformations used in the report are all based on rather crude assumptions. Essentially, this is the case with Captain Gaillard's assumption for transforming wave energy into equivalent energy of flowing water.<sup>2</sup> The tests of S. Ishbash for the stability of stones in flowing water are not strictly applicable to the case. Mr. Ishbash made tests for building cofferdams by dumping rocks into flowing water.<sup>5</sup> A stone already at rest in a fill has a different condition of exposure to flowing water, and its mass is at rest. The Ishbash deduction applied to riprap is therefore an approximation only.

The lack of specific tests is felt most severely in reference to the filter quality of the riprap. Suppose an outside layer of uniform stones has been established, the size of the stones having been selected to withstand a specific wave. What will be the forces passing this first screen? What will pass the second screen? Only by knowing the answers to these questions can the thickness and the desirable grading of riprap be established.

In soil mechanics filter grading limits have been established. However, they apply to seepage in saturated soil and are not meant to deal with dynamic forces. For riprap design the characteristics of a hydraulic energy filter will first have to be established.

The design procedure outlined in the report has been used by the Tennessee Valley Authority (TVA) as a basis for makeshift design since 1940. When the procedure was assembled, the best data then readily available were used. There was no misconception about their shortcomings. It was simply felt that the data offered at least a reasonable guide to adequate proportioning, which is far superior to no guide at all.

In TVA practice this method has been applied to the slope protection design for six earth embankments. The total length of these structures is 4.2 miles. The area covered by riprap on both the headwater and the tailwater side amounts to 77 acres. Since their construction these areas have been inspected periodically. No failure has been observed (as of 1948) at any place. However, this statement does not prove much. Although moderate storms have occurred, there is no record of any storm having approached the intensities assumed for design.

More interesting observations on behavior can be reported from another type of application of the same method. Below a dam the natural slopes of the river channel are usually exposed to waves far greater than they were before the construction of the dam. The increase, of course, depends on the type of spillway and dissipating arrangement used for the flood flows. These slopes

<sup>2</sup>"Wave Action in Relation to Engineering Structures," by D. D. Gaillard, *Professional Paper No. 31*, Corps of Engrs., U. S. Army, U. S. Govt. Printing Office, Washington, D. C., 1904.

<sup>5</sup>"Construction of Dams by Dumping Stones Into Flowing Water," by S. Ishbash, Leningrad, 1932.

usually must be protected to be stable under the new exposure. In many cases where access roads, switchyard fills, or navigation dikes are involved, this protection becomes an essential feature. The method outlined in the subcommittee report was used for dimensioning the riprap at the discharge channels below ten different dams. At these locations the destructive forces originate from three sources—river currents, wind-induced waves, and waves induced by falling water during spilling operation. In the vicinity of the dissipating area, this latter source usually produces by far the biggest component. For this component the wave characteristics in each case have been measured during model operation in the hydraulic laboratory. In most cases the controlling wave pattern for design was produced by the "design flood." No floods approaching these discharges have occurred at any of these places, but at two locations protection was applied to fill slopes, the top of which is far below the stage of the design flood. Discharges reaching the top of these fills have occurred since construction, and actual observations under exposure to design conditions could be made. One of these cases is at Watts Bar (Tennessee), where the fill below the dam forms a dike to separate the navigation channel from the river channel. This fill rises to the flood stage, at which navigation becomes impracticable. Design was based on waves emanating from the stilling basin at a discharge which produces waves that just overlap the dike. On two occasions the flood stage has reached within about a foot of this level. The riprap did not show any signs of failure.



FIG. 12.—DIKE AT WATTS BAR, TENNESSEE, WITH THE RIVER DISCHARGING 193,000 CU FT PER SEC

Fig. 12 shows a view of this dike with the river discharging 193,000 cu ft per sec. (The design for the slope protection was based on a discharge of 210,000 cu ft per sec.) The picture was taken from the spillway deck of the dam. The slope protected by riprap is in the background. The slope in the first 300 ft downstream from the stilling basin is protected by a concrete pavement. This change in type of construction was made necessary because the

required weights of stone near the stilling basin were computed as 9,000 lb, which was far greater than could be produced and handled economically at the Watts Bar quarry. Therefore, a highly articulated, interlocked concrete pavement 15 in. thick was used at the zone of highest waves. The riprap starts where the computed weight of stones was 2,700 lb, a size that could be produced and placed economically with the equipment on hand. It may be mentioned that at another TVA dam stones weighing 12,000 lb were placed adjacent to the dissipating area.

The other case affording observations of value is the slope protection along the flood plain below Kentucky Dam (Kentucky). A number of floods have reached the flood plain. However, because the discharge channel is affected by backwater from the Ohio River, not all these floods produced the wave exposure which had been assumed for the design. These waves are caused by the floodwaters spilling over the dam. Since the reservoir was filled, the discharge corresponding to the design condition used for the slope protection has occurred four times. These floods caused some damage to the pavements of the roads located on the flood plain, but the slope itself remained stable and the riprap did not show any defects after exposure.

From these six tests under "design" exposure, it may be concluded that the method gives safe results. However, there is no way of judging what the remaining safety margin was at the time of these exposures. Was the protection just barely stable, did it have a reasonable safety margin left, or was it uneconomically overdesigned? Only tests that would clarify the fundamental relations could answer these questions.

As was already mentioned, no information on the action of riprap as a filter for hydraulic energy was available when the rules for thickness and grading of the riprap had to be made. It was considered necessary to provide depth enough to place the heavier stones near the surface and some lighter ones below. As an expedient, the depth of riprap (that is, the distance between the exposed surface and the top of the underlying blanket of gravel or crushed rock) was specified to be equal to or larger than the diameter of stone obtained from Eq. 7, which assumes a spherical shape. Because the shape of the stone is usually prismatic, this depth specification gives space for the large stone plus some smaller ones. The placing of the large stones near the surface was encouraged. Specifications required that 50% of the stones were to be of a weight equal to or more than the weight computed from the Ishbash formula.

The subcommittee has indicated its full awareness of the meagerness of reliable data and of the economic importance of the problem involved. The writers hope that the subcommittee will be able to sponsor a research plan which will ultimately provide the fundamental physical data on which a reliable design of slope protection can be based.